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No. 7
V. 32

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Coming next month in the JOURNAL

MATTHYS P. LEVY discusses the design of and construction procedure for a "Precast Grid Wall" comprised of closely spaced precast concrete columns. The cost of a grid wall is compared to cost of conventional curtain wall with separate structural columns, and factors affecting cost are evaluated.

A. H. MATTOCK, L. B. KRIZ, and EIVIND HOGNESTAD offer an ultimate strength design theory, characterized by simplicity without significant loss of accuracy, in their paper, "Rectangular Concrete Stress Distribution in Ultimate Strength Design."

"Creep of Prestressed Concrete Beams" by W. C. COTTINGHAM, P. G. FLUCK, and G. W. WASHA presents test results obtained by the trio during 7 years of sustained loading of six prestressed concrete beams.

E. R. CANCIO and A. HERRERA explain solution adopted for the structural design of an industrial building in Havana, Cuba, in "Prestressed Precast Concrete Arches Span 102 ft 6 in. for Industrial Roof." Prefabrication was used to a large extent. Unit costs are also furnished.

J. GLUCKLICH and O. ISHAI present the results of "Further Study of the Rheological Behavior of Hardened Cement Paste Under Low Stresses."

"Ultimate Strength of a Folded Plate Structure," by GREGORY P. CHACOS and JOHN B. SCALZI reports on structural research conducted to determine the behavior of the folded plate structure as a simple beam with an irregular cross section; also to verify the ultimate moment capacity by the rectangular stress block method.

On the Cover—Bridge over Chambal River, Rajasthan, India, is one of the longest submersible bridges in the world. The superstructure consists of nine reinforced concrete parabolic arches over the deepest portion of the river, flanked by rigid frames reaching to the river banks. Further details are given in the News Letter.—Photo courtesy The Indian Concrete Journal, Bombay, India.

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

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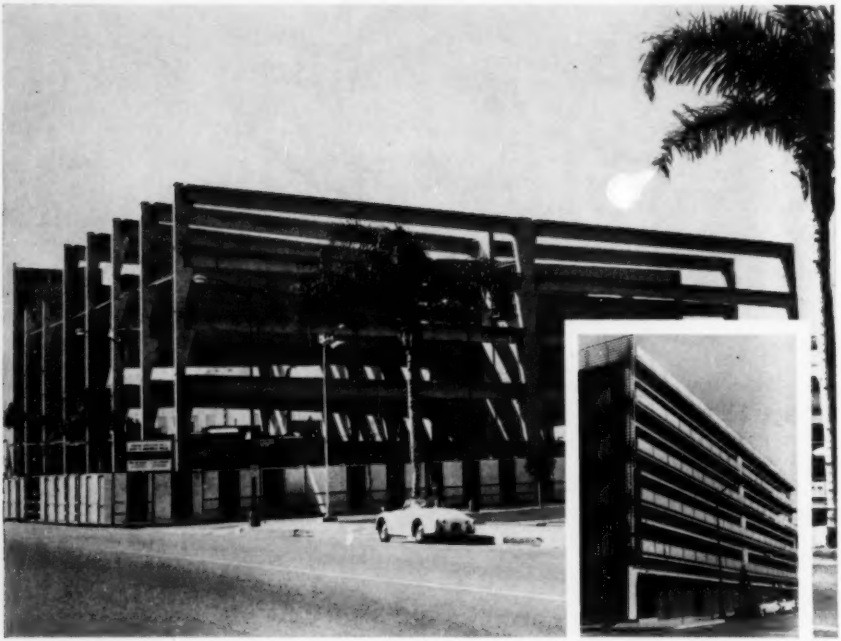

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Ultimate Strength of Nonrectangular Structural Concrete Members

By ALAN H. MATTOCK and LADISLAV B. KRIZ

Test data are presented which demonstrate the applicability to nonrectangular structural concrete members of fundamental plasticity concepts deduced from tests of rectangular structural concrete members.

Flexural tests are reported of symmetrical beams with triangular-shaped concrete compression zones, and of unsymmetric T-beams.

The behavior of the beams tested, and in particular their ultimate strength, is compared with the behavior and ultimate strength predicted by two theories derived from previous tests of rectangular members. One of the two theories considered is of a general nature and is intended primarily for research purposes, the other is based on an equivalent rectangular stress distribution and is suitable for practical design.

■ STRUCTURAL CONCRETE DESIGN frequently involves members subject to bending about two axes, or other situations in which the concrete compression zone becomes nonrectangular in shape. It is, therefore, important to know whether ultimate strength design theories based on data obtained from tests of members with rectangular-shaped compression zones are directly applicable to members with nonrectangular compression zones.

Test data are scarce for structural concrete members other than those having rectangular compression zones. Hognestad[1] reported tests of 30 eccentrically loaded circular columns all of 12-in. diameter and reinforced with 8 #7 bars. The variables were eccentricity of load and strength of concrete. Anderson and Lee[2] reported tests of ten square section columns of 4-in. side, subject to axial thrust and equal bending about the two major axes of the column cross section. The amount of reinforcement in the column was the only variable in these tests. Although good agreement was found[3] between experimental and calculated ultimate strength for the above tests, using an equivalent rectangular stress distribution theory based on tests of members with rectangular compression zones, it was considered desirable to obtain additional data from tests of members with nonrectangular compression zones. The experimental work described below was therefore carried out at the Portland Cement Association Laboratories during 1959.

EXPERIMENTAL WORK

Objectives and scope

(1) To obtain additional test data from tests of structural concrete members having nonrectangular compression zones.

(2) To use the above data to check the applicability, to members with nonrectangular compression zones, of two ultimate strength theories based on tests of members with rectangular compression zones.

This investigation was limited to beams in which the concrete compression zone at failure was triangular in shape. Concrete strengths used ranged from 1700 psi to 7000 psi. The ultimate strengths of half the beams tested were controlled by crushing of the concrete, and of the other half by yielding of the steel.

The observed behavior of the beams was compared with the behavior predicted by two ultimate strength theories outlined in the appendix to this paper. Both theories are based on data obtained from tests of structural concrete members with rectangular compression zones. The first theory is of a perfectly general character, particularized by the use of the concrete stress-strain curves obtained by Hognestad, Hanson, and McHenry[4] in a previous investigation carried out in the PCA Laboratories. This theory was previously applied to rectangular cross

TABLE 1 — PROPERTIES OF SERIES T-BEAMS

Beam No.	Concrete cylinder strength, f_c', psi	Reinforcement area, A_s, sq in.	Reinforcement yield stress, f_y, ksi
T-1	3620	0.88	49.0
T-2	3455	1.32	49.0
T-3	6290	1.80	45.0
T-4	1735	0.88	49.5
T-5	3500	1.32	50.5
T-6	7000	1.80	45.0

TABLE 2 — PROPERTIES OF SERIES L-BEAMS

Beam No.	Flange width, b, in.	Effective depth, d, in.	Concrete cylinder strength, f_c', psi	Reinforcement area, A_s, sq in.	Reinforcement yield stress, f_y, ksi
L-1	8	13.8	2820	1.20	45.6
L-2	8	13.8	6200	1.20	46.1
L-3	8	12.9	3260	2.40	45.6
L-4	8	12.9	5570	2.40	51.0
L-5	12	13.8	2050	1.20	47.9
L-6	12	13.8	6320	1.20	47.5
L-7	12	12.9	2225	2.40	43.0
L-8	12	12.9	6420	2.40	43.0
L-9	16	13.8	2680	1.20	45.4
L-10	16	13.8	4820	1.20	44.5
L-11	16	12.9	2530	2.40	50.7
L-12	16	12.9	5820	2.40	50.5

Alan H. Mattock (ACI member) joined the staff of the Portland Cement Association Research and Development Laboratories, Skokie, Ill., in 1957. After graduation from London University he served with the Fire Research Station at Elstree, England, and in the Lancashire County Bridgmaster's design office. He returned to study and was awarded MS (Eng), London University, in 1949. For the next 3 years he was a district engineer in British Guiana, South America, where the principal work was the construction of sea and river walls, and drainage and irrigation works.

From 1952 to 1957 Dr. Mattock was a lecturer at Imperial College, University of London, having charge of the Concrete Technology Laboratory; while there he carried out research and consulting work, and in 1955 was awarded PhD, London University.

Ladislav B. Kriz (ACI member) graduated in civil engineering from the University of Illinois in 1957, with previous schooling at the Technical University in Prague, Czechoslovakia, and at the School of Civil Engineering in Madrid, Spain. His undergraduate studies were interrupted by brief periods of employment as draftsman with Zublin-Perriere, Paris, France; as a designer with Byrnes Associates, New York, N.Y.; and a 4-year tour of duty as surveyor in the U. S. Air Force.

He joined the staff of the Structural Section of the Portland Cement Association Research and Development Laboratories in 1957. During his employment with the association, he earned his MS degree from Northwestern University.

sections.[5,6] The second theory is based on the use of an equivalent rectangular distribution of the concrete compressive stresses. The development of this theory is described in detail by the authors in another paper.[3]

Materials

For convenience in scheduling the tests, the concretes of 5000 psi strength or over were made using Type III portland cement, while the lower strength concretes were made using Type I portland cement. The coarse aggregate was a gravel of ¾-in. maximum size and the fine aggregate was Elgin sand. The slump varied from 1 to 3 in. and an air-entraining agent was added to provide from 4.5 to 5.5 percent air in the concrete. Moist curing under burlap took place for the first 3 days after casting. Subsequent storage was at 73 F and 50 percent relative humidity. Concrete cylinder strengths given in Tables 1 and 2 are the average of at least four 6 x 12-in. cylinders taken from the batch of concrete placed in the compression zone of the beams.

All reinforcing steel used conformed to ASTM A 305-53T for deformations, and had yield points as listed in Tables 1 and 2. Stirrups were provided at 3-in. centers outside the region of constant moment in both series of beams. The stirrups were #3 deformed bars with a yield point of 50 ksi.

Test beams

Series T—The six beams of this series had the shape shown in Fig. 1. They were designed with a triangular compression zone in the failure region, and were of symmetrical form to simplify loading arrangements.

The variables in this series were concrete strength and amount of tensile reinforcement. The relevant data are contained in Table 1.

The areas of reinforcement were chosen so as to ensure that ultimate strength would be controlled by concrete compression. This was done in order that assumptions as to the shape of the concrete stress-strain curve, etc., made in the ultimate strength theories, should be critical.

Series L—The beams of this series have an unsymmetric T, or inverted L, cross section. If this type of beam is loaded in the plane of the web center line, and no lateral restraint is present, then the neutral axis will be inclined, producing a nonrectangular compression zone.

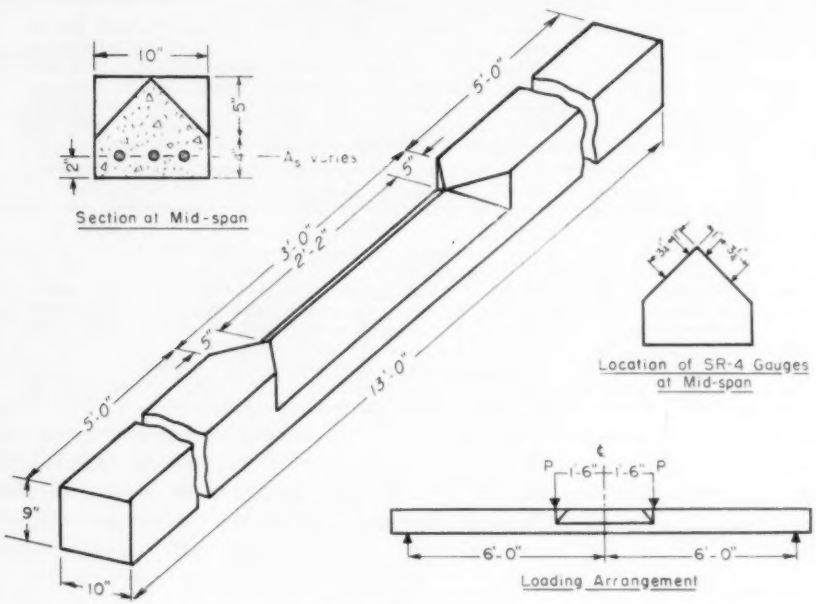

Fig. 1—**Beams with triangular compression zones (Series T)**

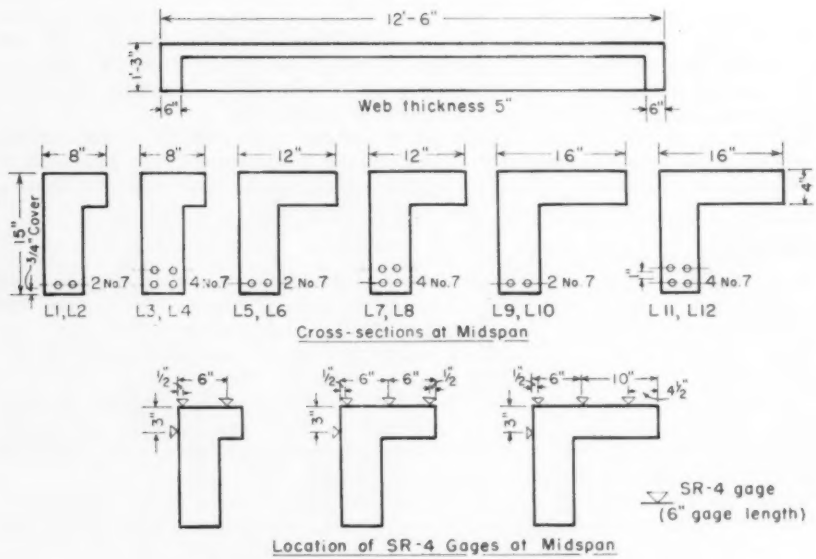

Fig. 2—**Unsymmetric T-beams (Series L)**

The variables in this series were concrete strength, amount of tensile reinforcement, and width of beam flange. The particulars of individual beams are contained in Fig. 2 and Table 2.

The reinforcement areas were chosen so that half of the beams would have ultimate strength controlled by yield of the steel, as is considered desirable in practical design. The other half were designed for approximately balanced conditions at ultimate strength. This was done in order that assumptions as to the shape of the concrete stress-strain curve, etc., made in the ultimate strength theories, should be critical for this group of beams.

Arrangements for tests

Series T—The beams were supported on rollers at 12-ft centers and were loaded at two points 18 in. either side of the midspan. This is shown diagrammatically in Fig. 1. The load was applied by two calibrated 20,000-lb hydraulic rams operating below the laboratory test floor. A system of cross heads and pull rods transmitted the loads to the beam, as may be seen in Fig. 3. The oil pressure was supplied to the rams and measured by a Riehle "Pendomatic" mobile pumping and indicating unit. The details of this hydraulic equipment have been described elsewhere.[7]

Concrete strains in the compression zone were measured at the locations indicated in Fig. 1. The rows of 6-in. gage length SR-4 gages extended over the entire length of that part of the beams subject to constant bending moment. The strains were recorded continuously during the tests using six-channel Sanborn 67A recorders, as seen in Fig. 3.

Series L—These beams were also supported on rollers at 12-ft centers and were loaded at two points 18 in. either side of midspan. The loads were applied to the beams in the plane of the center line of the web through ball seatings. These were designed to permit relative rotation of the beam and the loading cross heads as the beam twisted under load. The loads were applied by two

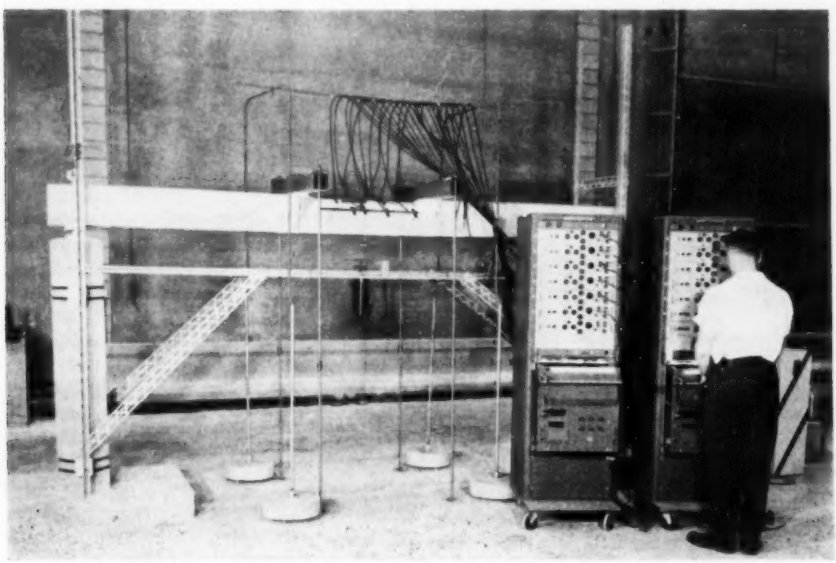

Fig. 3—Arrangement for test of beams (Series T)

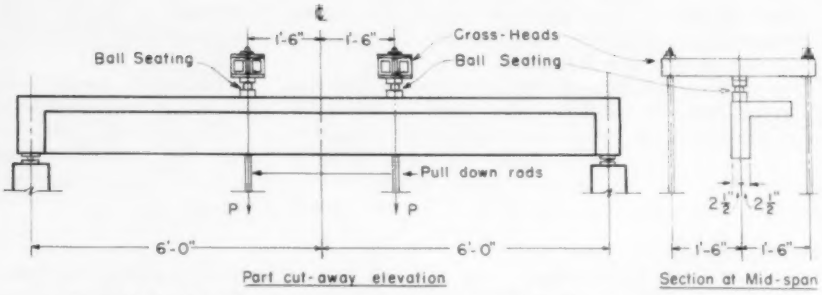

Fig. 4(a)—Loading arrangement, unsymmetric T-beams (Series L)

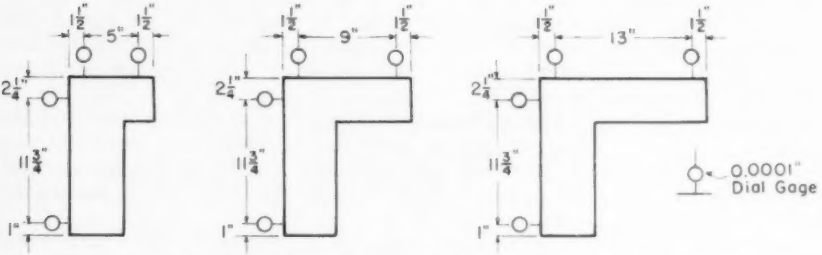

Fig. 4(b)—Location of deflection gages at midspan and at supports, unsymmetric T-beams

pairs of 20,000-lb calibrated rams positioned under the test floor and acting through a system of cross heads and pull rods, as may be seen in Fig. 4(a) and Fig. 5. The beams were purposely placed on high columns to increase the length of the pull rods and so decrease the possibility of lateral restraint by the loading equipment of the beam under test. The oil pressure was supplied to the rams and measured as for Series T.

Concrete strains were measured over the entire center 3 ft of the beams by rows of SR-4 gages located as shown in Fig. 2. The strains were recorded continuously during the tests by use of Sanborn 67A recorders which may be seen in Fig. 5.

Vertical and horizontal deflections were measured using twelve 0.001-in. dial gages mounted on a bridge of Dexion angle as seen in Fig. 5. The gages were located as shown in Fig. 4(b). This arrangement of gages enabled twisting of the beam under load to be measured.

Test procedure

In both series the load was applied in increments, the size of the increment depending on the estimated ultimate strength of the beam under test. A continuous record of strains was made by the Sanborn recorders, so that the strain behavior of the beams was known up to the instant of failure. At each stage of loading the cracking behavior of the beams was observed, and in the case of Series L, readings were taken from the deflection dial gages. The time taken for each test is given in Tables 3 and 7.

In every case loading was continued until the beams developed their flexural ultimate strength. Zones of maximum moment of three typical beams of Series L are shown in Fig. 6 after loading of the beams to destruction.

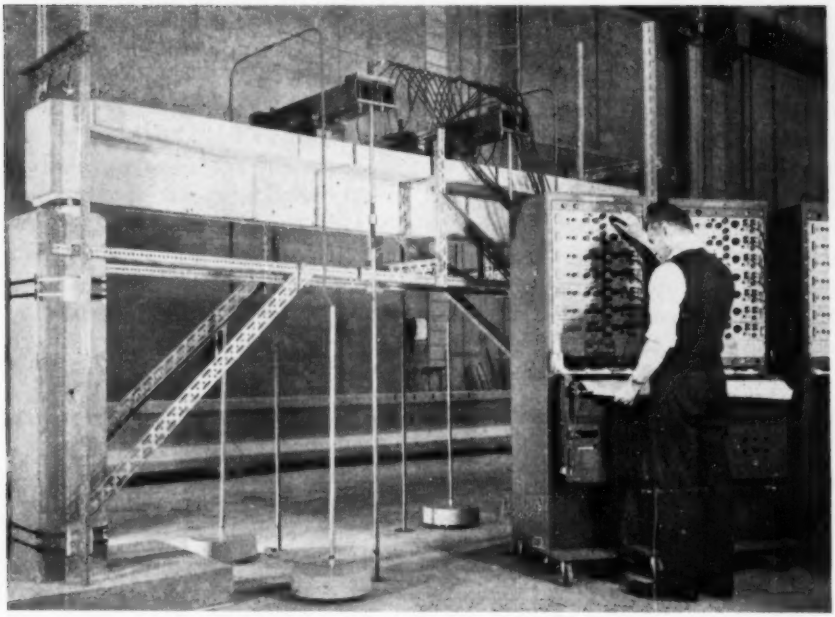

Fig. 5—Arrangement for test of beams (Series L)

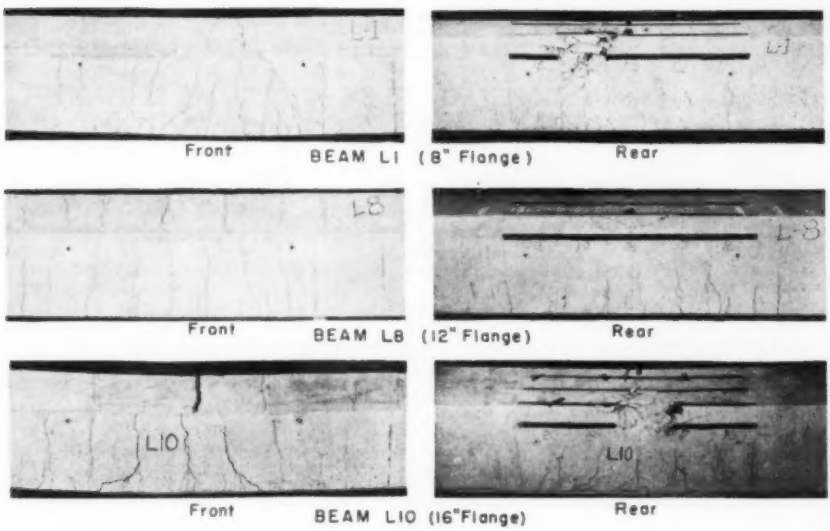

Fig. 6—Typical beams (Series L) after test

COMPARISON OF OBSERVED AND CALCULATED BEHAVIOR

For the purpose of comparing the observed and calculated performance, the behavior and ultimate strengths of the members are calculated using two separate theories. The first is based on a general form of concrete stress-strain curve, and is referred to as the General Theory. The second is based on the use of an equivalent rectangular concrete stress distribution, and is referred to as the Equivalent Rectangular Stress Distribution Theory. The first theory may be used to trace the behavior of the members under progressively increasing load, culminating with the ultimate strength of the members. The second theory is of a form more suitable for practical design and will yield only the ultimate strength of the members. Both the above theories are developed in detail in the Appendix.

Series T—Symmetric beams with triangular compression zones

Deformations—The maximum concrete compressive strain and the depth of neutral axis obtained from measured concrete strains are plotted in Fig. 7(a), (b), and (c). Also plotted in these figures are the theoretical values of the same quantities, calculated using the two theories described in the appendix to this paper.

It can be seen that the general theory predicts almost exactly the variation of maximum concrete strain with bending moment for almost the whole range of applied moments. The calculated value of maximum concrete strain at ultimate strength (Table 3) is not quite so close to the observed value as are the calculated values at lower applied moments. However, this is not of importance in the case of these beams since the maximum concrete strain increases rapidly for very little increase in resisting moment at loads approaching ultimate, hence

TABLE 3—MAXIMUM CONCRETE STRAIN AT ULTIMATE STRENGTH—SERIES T

Beam No.	Maximum concrete strain at ultimate strength, ε_c, in. per in.			Observed ε_c / Calculated or assumed ε_c		Total testing time, min
	Test	Calculated general theory	Assumed equivalent rectangular stress distribution theory	General theory	Equivalent rectangular stress distribution theory	
T-1	0.0037	0.0032	0.0030	1.16	1.23	26
T-2	0.0038	0.0040	0.0030	0.95	1.27	30
T-3	0.0035	0.0034	0.0030	1.03	1.17	46
T-4	0.0031	0.0040	0.0030	0.78	1.03	27
T-5	0.0040	0.0040	0.0030	1.00	1.33	39
T-6	0.0038	0.0032	0.0030	1.19	1.27	50
			Average	1.02	1.22	
			Standard deviation	0.149	0.106	

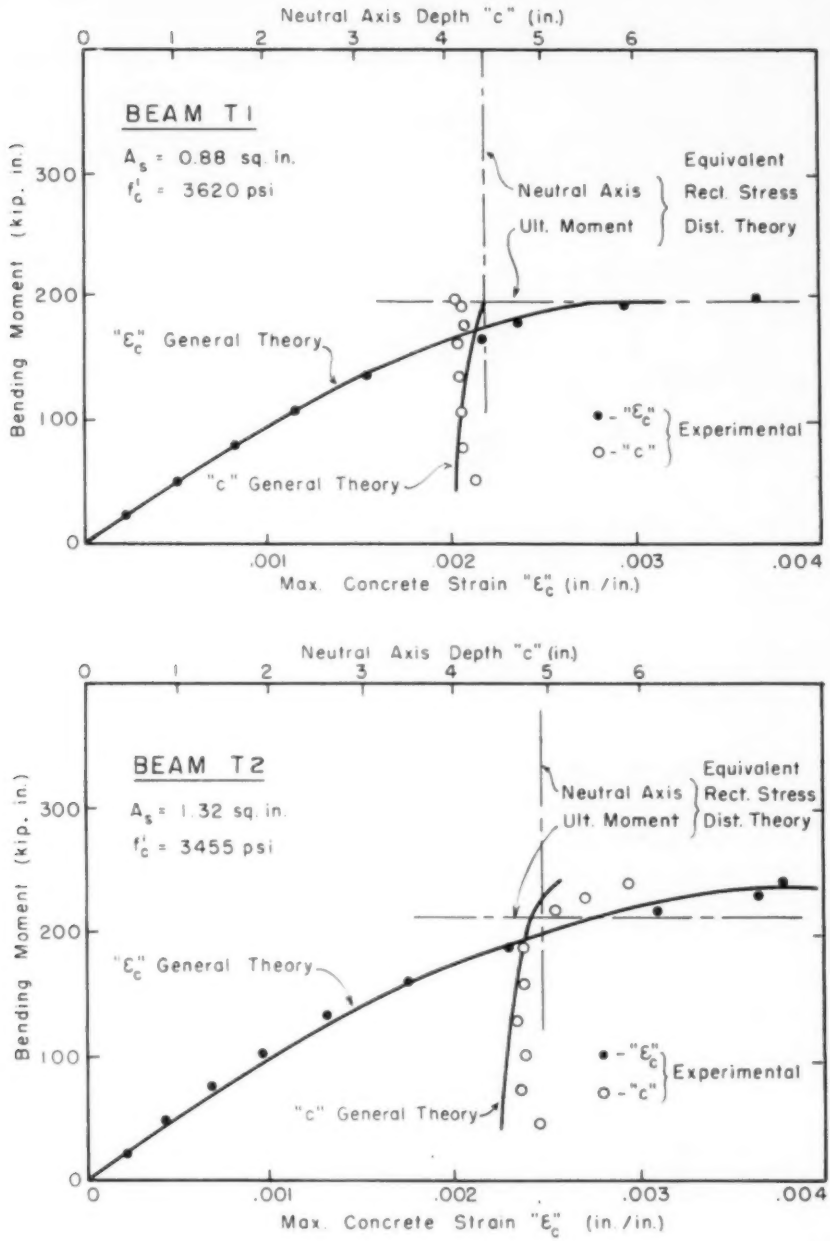

Fig. 7(a)—Variation of neutral axis position and maximum concrete strain with applied bending moment, Beams T1 and T2

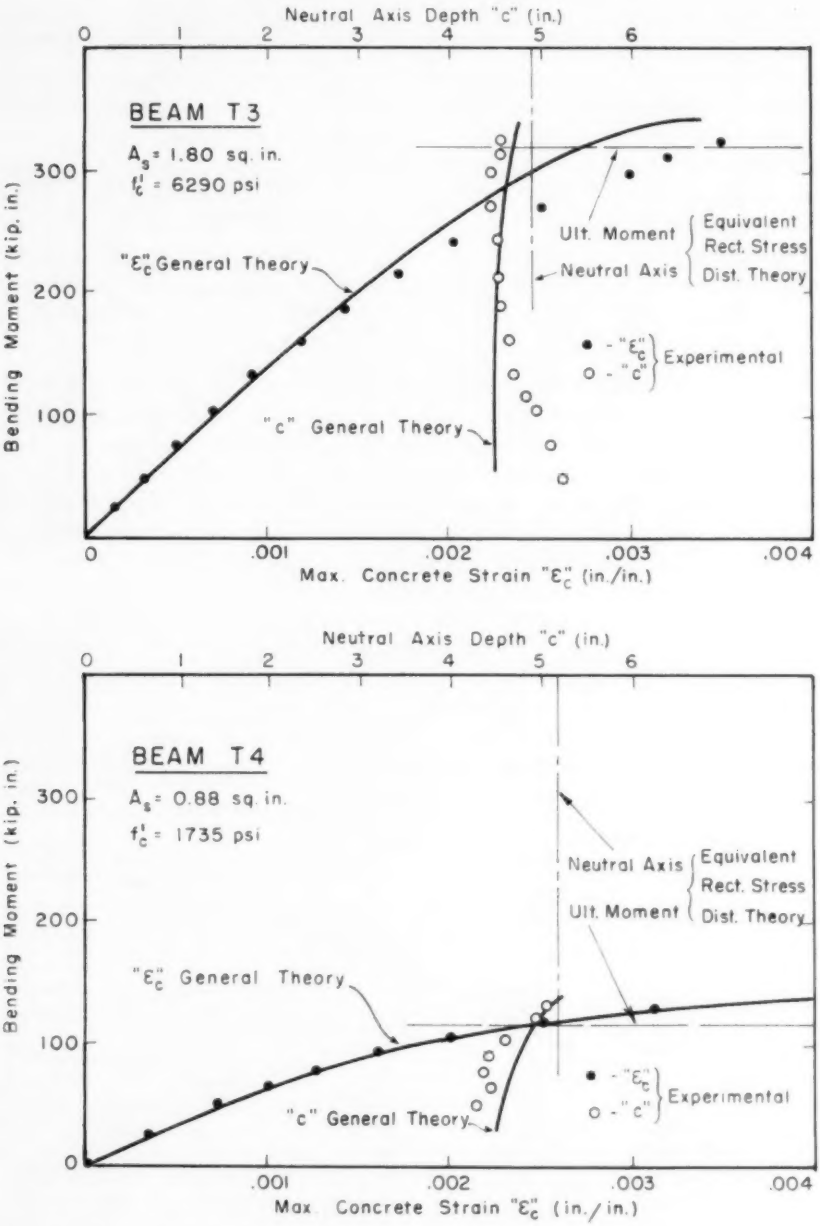

Fig. 7(b)—Variation of neutral axis position and maximum concrete strain with applied bending moment, Beams T3 and T4

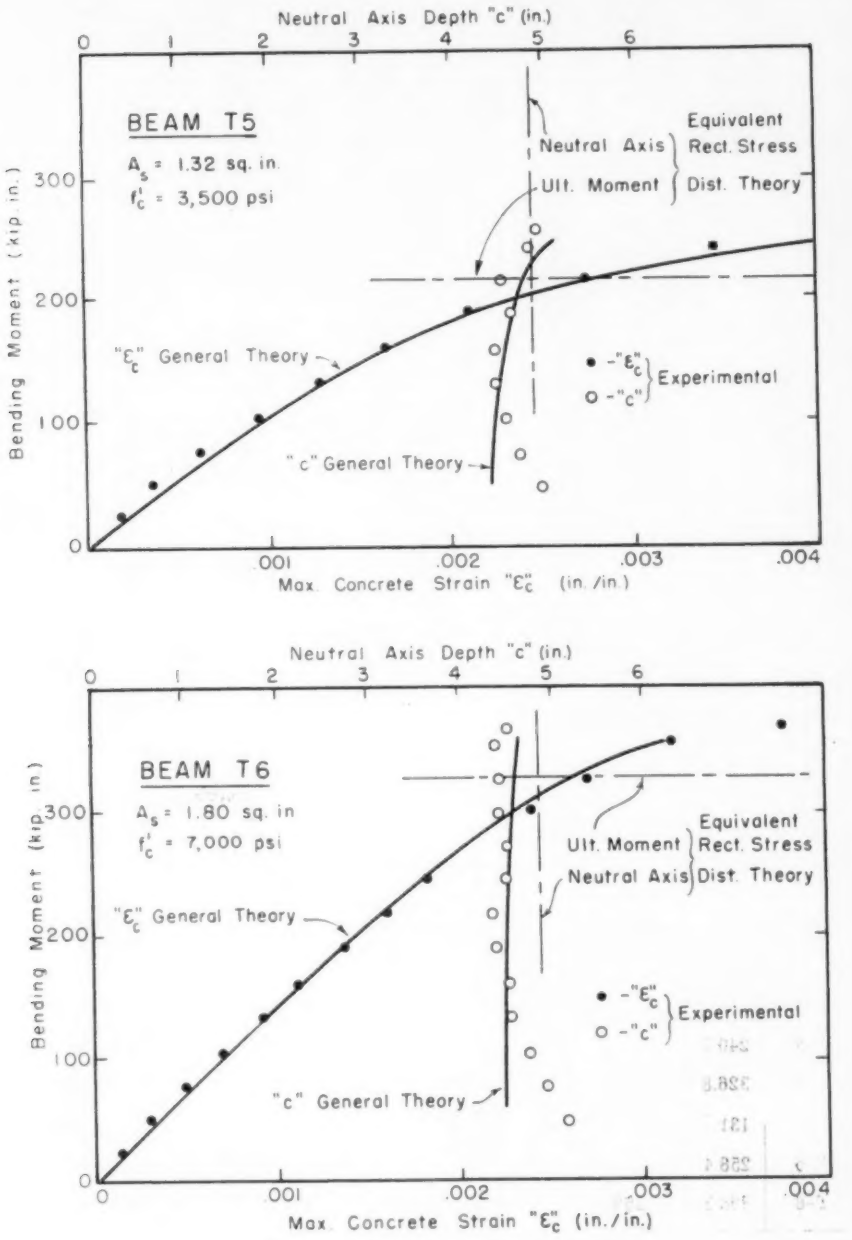

Fig. 7(c)—Variation of neutral axis position and maximum concrete strain with applied bending moment, Beams T5 and T6

any error in the estimation of maximum concrete strain leads to a much smaller error in the estimation of ultimate strength.

The location of the neutral axis position using the same theory is close to the experimentally observed position. At low bending moments the influence of tensile stresses in the concrete over an extensive zone is reflected in the discrepancy between the calculated and observed neutral axis locations. In the theoretical calculation the concrete is assumed to have no tensile strength, the calculated location of the neutral axis is therefore above the observed location at low loads. Confirmation of this hypothesis is provided by the behavior of Beam T-4 in which shrinkage cracks existed in the tensile zone at the time

TABLE 4 — NEUTRAL AXIS—SERIES T

Beam No.	Neutral axis depth c at ultimate strength, in.			Observed N. A. depth / Calculated N. A. depth	
	Test	Calculated general theory	Calculated equivalent rectangular stress distribution theory	General theory	Equivalent rectangular stress distribution theory
T-1	4.08	4.32	4.40	0.95	0.93
T-2	5.82	5.05	4.90	1.15	1.19
T-3	4.57	4.79	4.90	0.96	0.93
T-4	5.01	5.20	5.16	0.96	0.97
T-5	4.96	5.04	4.90	0.98	1.01
T-6	4.57	4.66	4.90	0.98	0.93
			Average	1.00	0.99
			Standard deviation	0.075	0.101

TABLE 5 — ULTIMATE STRENGTH—SERIES T

Beam No.	Ultimate moment, kip-in.			M_{test}/M_{calc}		Type of failure
	Test	Calculated general theory	Calculated equivalent rectangular stress distribution theory	General theory	Equivalent rectangular stress distribution theory	
T-1	196.8	196.3	194.5	1.00	1.01	Balanced
T-2	240.9	243.1	215.0	0.98	1.12	Compression
T-3	326.8	346.3	320.0	0.92	1.02	Compression
T-4	131.8	141.0	116.3	0.93	1.13	Compression
T-5	258.4	245.0	219.0	1.05	1.18	Compression
T-6	368.3	358.6	323.0	1.03	1.14	Compression
			Average	0.98	1.10	
			Standard deviation	0.050	0.069	

of test. At moments approaching the ultimate flexural strength of the beams the uncracked concrete tensile zone becomes insignificant, resulting in close agreement between calculated and observed neutral axis locations as seen in Table 4.

The equivalent rectangular stress distribution theory enables only the calculation of the ultimate strength of the members and the location of the neutral axis at ultimate strength. It can be seen in Fig. 7(a), (b), (c), and Table 4 that the neutral axis positions calculated using this theory are in fairly close agreement with the observed positions. In Table 3, the assumed maximum concrete strain at ultimate strength, 0.003 in. per in., is compared to the observed value and found to be reasonably conservative.

Ultimate strength—Both theories lead to a close estimate of the ultimate strength of the beams of this series, as may be seen in Table 5. Application of the general theory resulted in a value of M_{test}/M_{calc} more nearly equal to one, and also to a smaller standard deviation, than that obtained through use of the equivalent rectangular stress distribution theory. However, the accuracy of this last theory is still acceptable and any errors appeared to be in the direction of under-estimation of ultimate strength.

Series L—Unsymmetric T-beams

Deformations—The maximum concrete strain and location of neutral axis for a typical beam of this series are plotted in Fig. 8. Since for these beams the neutral axis is inclined, two measurements are necessary to define its position. In this instance the two chosen are the inclination of the neutral axis to the horizontal, α, and the vertical distance, Y, measured from the top face to the intersection of the neutral axis and the rear vertical face of the beam. Also plotted in Fig. 8 are the values of maximum concrete strain and the neutral axis location calculated using the two theories.

It can be seen that there is close agreement between the maximum concrete strains predicted by the general theory and the observed concrete strains. However, the agreement between observed and calculated neutral axis position is not very close at most low loads, but is good at ultimate strength. The discrepancy between the calculated and observed neutral axis positions at loads below ultimate is again due to neglect of concrete tensile strength in the theoretical analysis. Before cracking of the concrete at the bottom of the beam, a considerable resultant concrete tensile force pulls the neutral axis down below the calculated position. After cracking at the bottom of the beam, but before cracking of the flange, the resultant concrete tensile force in the lower outer portion of the flange causes the neutral axis to be less steeply inclined than the calculated position which neglects this tensile force. At ultimate strength, however, cracking is well de-

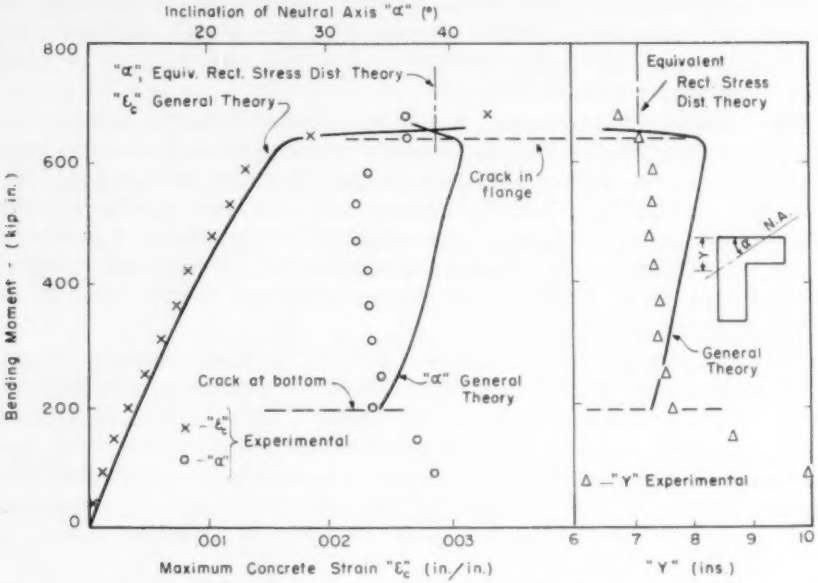

Fig. 8—Variation of maximum concrete strain, inclination of neutral axis, and location of intersection of neutral axis and rear vertical face of beam, Y, for typical unsymmetric T-beam (L1)

veloped in both web and flange, and the calculated and observed neutral axis positions agree well as may be seen in Table 6 and Fig. 9. The location of the neutral axis at ultimate strength, calculated using the equivalent rectangular stress distribution theory, is also in reasonably close agreement with the observed location.

The observed and calculated maximum concrete strains at ultimate strength for all beams of Series L are listed in Table 7. As with the beams of Series T, the agreement between calculated and observed values of maximum concrete strain is not as good at ultimate strength as at lower applied moments.

It is of interest to compare the inclination of the neutral axis at various stages of loading for the three beams (L1, L5, and L9) which are of similar quality concrete and identical reinforcement area. The only major variable in this group is the width of the beam flange. In Fig. 10, tan α, the tangent of the angle of inclination of the neutral axis, as obtained from strain measurements, is plotted against the ratio of the flange width to the web width, b/b'. It can be seen that in the first two stages of loading, i.e., before cracking of the flange, the ratio b/b' has a considerable influence on the inclination of the neutral axis. However, in the final stage of loading close to ultimate strength, when cracking is well developed in both web and flange, the flange width

has no influence on the inclination of the neutral axis provided the flange is wide enough so that the neutral axis cuts through the top face. This is in agreement with the theoretical behavior predicted by both theories considered in this study.

Also plotted in Fig. 10 is the ratio of the measured horizontal deflection to the measured vertical deflection for the second stage of loading, i.e., after web cracking but before cracking of the flange. These

TABLE 6 — NEUTRAL AXIS—SERIES L

Beam No.	Distance, Y, in.			Inclination of neutral axis, a, deg		
	Test	General theory	Equivalent rectangular stress distribution theory	Test	General theory	Equivalent rectangular stress distribution theory
L-1	6.92	6.55	7.10	36.7	37.1	38.8
L-2	4.34	3.60	3.80	23.3	21.6	21.0
L-3	9.01	11.43	11.02	43.1	53.3	51.4
L-4	9.12	8.59	8.92	36.2	43.6	39.5
L-5	6.66	9.08	10.35	34.0	46.5	49.5
L-6	3.44	3.66	3.83	18.2	21.9	20.7
L-7	10.51	12.79	12.20	46.5	56.9	54.1
L-8	8.30	6.56	6.90	34.7	35.7	33.8
L-9	6.67	6.81	7.50	37.2	38.2	40.3
L-10	4.65	4.17	4.27	19.2	25.0	24.5
L-11	11.47	12.46	11.82	50.6	56.2	53.3
L-12	8.50	8.24	8.60	42.0	42.3	41.0

TABLE 7 — MAXIMUM CONCRETE STRAIN AT ULTIMATE STRENGTH — SERIES L

Beam No.	Maximum concrete compressive strain at ultimate strength, ϵ_c, in./in.			Observed ϵ_c / Calculated or assumed ϵ_c		Type of failure	Total testing time, min
	Test	Calculated general theory	Assumed equivalent rectangular stress distribution theory	General theory	Equivalent rectangular stress distribution theory		
L-1	0.0033	0.0032	0.0030	1.03	1.10	Tension	77
L-2	0.0033	0.0030	0.0030	1.10	1.10	Tension	71
L-3	0.0036	0.0036	0.0030	1.00	1.20	Compression	71
L-4	0.0032	0.0030	0.0030	1.07	1.07	Balanced	109
L-5	0.0039	0.0032	0.0030	1.22	1.30	Tension	115
L-6	0.0042	0.0030	0.0030	1.40	1.40	Tension	120
L-7	0.0029	0.0040	0.0030	0.73	0.97	Compression	60
L-8	—	0.0030	0.0030	—	—	Tension	100
L-9	0.0034	0.0032	0.0030	1.06	1.13	Tension	49
L-10	0.0040	0.0030	0.0030	1.33	1.33	Tension	60
L-11	0.0035	0.0040	0.0030	0.88	1.17	Compression	94
L-12	0.0035	0.0030	0.0030	1.17	1.17	Tension	81
			Average	1.09	1.18		
			Standard deviation	0.191	0.125		

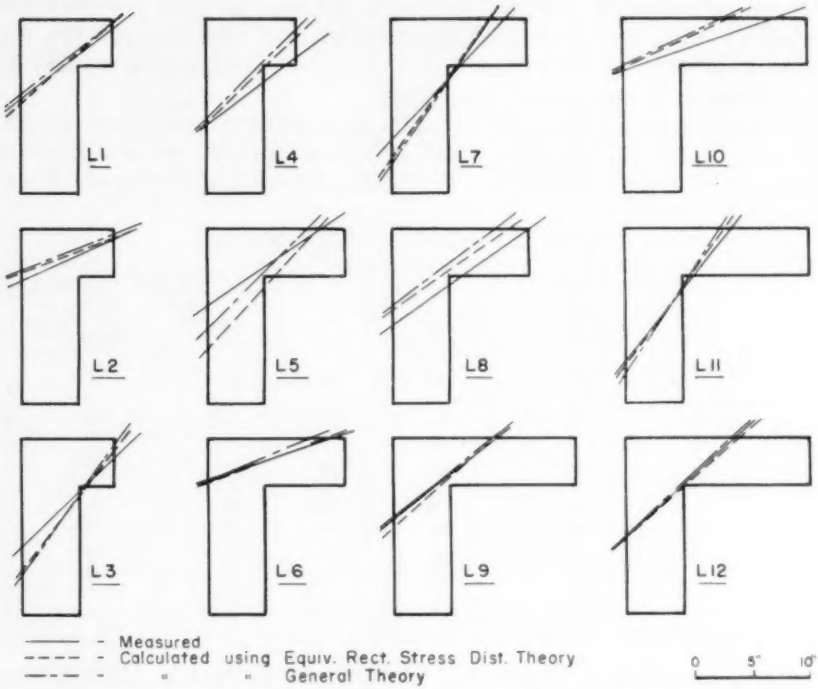

Fig. 9—Comparison of measured and calculated locations of neutral axis at ultimate strength (Series L)

deflection ratios are seen to be in almost exact agreement with tan α as determined by strain measurements at this same stage of loading. The ratios of deflection are taken from Fig. 11, in which horizontal deflection is plotted against vertical deflection for equal increments of 2 kips in the total applied load. The influence of flange width on the deflection pattern is, as is to be expected, similar to its influence on neutral axis inclination.

The measured twist in the beams due to variation in location of neutral axis along the length of the beam was found to be insignificant. The maximum twist was of the order of one tenth of a degree.

Ultimate strength—The observed and calculated values of ultimate strength are listed and compared in Table 8. The cases in which ultimate strength was controlled by tension, or in which a balanced condition existed at ultimate strength, show excellent agreement between observed and calculated values of ultimate moment. In one of the compression-controlled beams, L-11, the agreement was less satisfactory. The concrete in this beam is of low strength and therefore the average concrete compressive stress at ultimate strength will be under-estimated

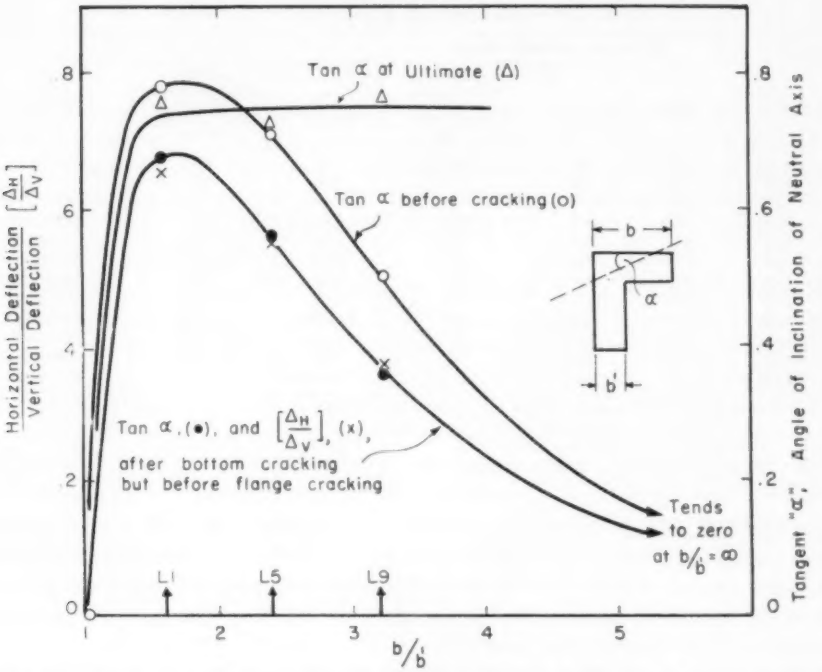

Fig. 10—Influence of Flange width/Web width on inclination of neutral axis and direction of deflection

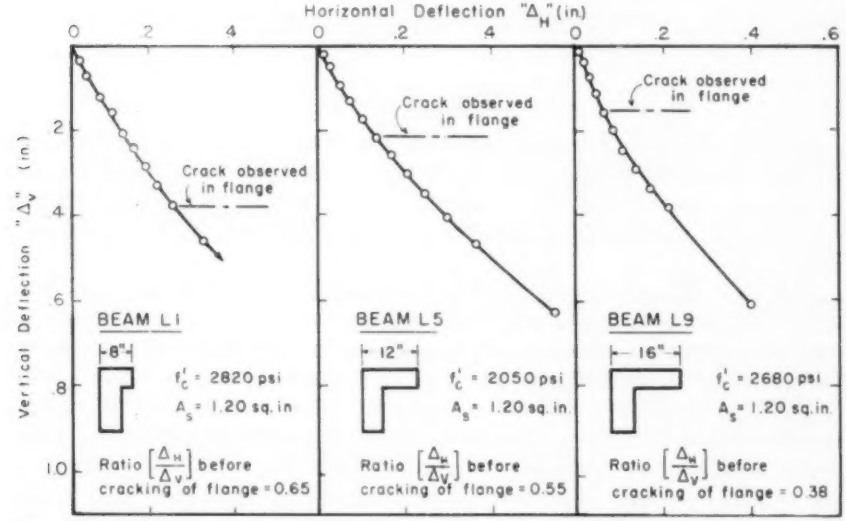

Fig. 11—Influence of flange width on deflection (deflections plotted for increments of 2 kips in total applied load)

TABLE 8 — ULTIMATE STRENGTH—SERIES L

Beam No.	Ultimate moment, kip-in.			M_{test}/M_{calc}		Type of failure
	Test	General theory	Equivalent rectangular stress distribution theory	General theory	Equivalent rectangular stress distribution theory	
L-1	678	652	647	1.04	1.05	Tension
L-2	693	709	710	0.98	0.98	Tension
L-3	1062	1043	952	1.02	1.12	Compression
L-4	1325	1288	1300	1.03	1.02	Balanced
L-5	671	642	627	1.05	1.07	Tension
L-6	778	729	733	1.07	1.06	Tension
L-7	876	805	695	1.10	1.26	Compression
L-8	1173	1146	1160	1.02	1.01	Tension
L-9	663	645	637	1.03	1.04	Tension
L-10	757	675	678	1.12	1.12	Tension
L-11	1129	885	775	1.27	1.46	Compression
L-12	1265	1288	1298	0.98	0.97	Tension
Average, all beams				1.06	1.10	
Average, tension control				1.04	1.04	
Standard deviation, all beams				0.078	0.138	
Standard deviation, tension control				0.043	0.047	

by the equivalent rectangular stress distribution analysis, which incorporates a maximum value of k_1k_3 based on 4000-psi concrete. The known conservatism of this method of analysis for low strength concrete beams failing by compression is also confirmed by the ratios of M_{test}/M_{calc} obtained for Beams L-7 and L-3.

It can be seen that for the beams in which ultimate strength was controlled by yield of the steel, the simplified analysis, using the equivalent rectangular stress distribution, gives nearly as accurate results as the general theory, using concrete stress-strain relationships obtained experimentally.[4]

CONCLUSIONS

In view of the agreement between observed and calculated behavior of both series of beams, and of the agreement between observed and calculated behavior of the columns of Hognestad[1] and Anderson and Lee[2] found in the authors' paper[3] previously referred to, it is concluded that:

(1) The ultimate strength theories considered, which are based on data obtained in tests of structural concrete members having rectangular concrete compression zones, are applicable to structural concrete members having nonrectangular concrete compression zones.

(2) The simplified ultimate strength theory, using an equivalent rectangular distribution of stress in the concrete compression zone, yields estimates of ultimate strength which are sufficiently accurate for design purposes. These are also close to the estimates of ultimate strength calculated using the general theory, which involves the use of actual measured concrete stress-strain curves.

APPENDIX

THEORETICAL ANALYSES

Two theoretical analyses are presented for both types of beams tested. The first is based on a general form of concrete stress-strain curve, and will be referred to as the General Theory. The second is based on the use of an equivalent rectangular concrete stress distribution instead of an actual curvilinear stress distribution. This analysis will be referred to as the Equivalent Rectangular Stress Distribution Theory.

In the general theory, the moment of resistance of the structural concrete section is expressed as a function of the concrete compression strain at the extreme edge of the section. This function is differentiated with respect to the strain to obtain the maximum value of the resisting moment, i.e., the ultimate strength. Alternatively, if it is desired to trace the behavior of the member under progressively increasing load, the resisting moment may be calculated for successively increasing values of extreme edge strain. It is then possible to pick out the maximum resisting moment by inspection. The calculation of resisting moment may also be programmed so as to yield the location of the neutral axis. This latter approach is particularly suitable for electronic computation.

The equivalent rectangular stress distribution theory utilizes certain simplifying assumptions, such as the equivalent rectangular stress distribution and a limiting value of concrete strain, and is not suitable for tracing the behavior of a section under increasing load. It has been found, however, to give reliable estimates of the ultimate strength of members for practical design purposes.

Notation

A = area under concrete stress-strain curve

A_s = cross-sectional area of tensile reinforcement

a = depth of equivalent rectangular stress distribution

b = width of flange of T-beam, or width of a triangular compression zone distance x from its base

b' = width of web of a T-beam

b_o = width of base of a triangular compression zone

C = total concrete compressive force

c = neutral axis depth, i.e., height of triangular compression zone

d = effective depth of beams

E_s = modulus of elasticity of reinforcing steel

f = concrete stress corresponding to strain ε

f_{av} = average concrete stress on compression zone

f_c' = concrete cylinder strength

f_y = yield strength of tensile reinforcement

I = second moment of area A about the f axis

k_1 = a/c

k_2' = ratio of the perpendicular distance from the apex of the triangular compression zone to the center of action of the resultant compressive force, to the depth c of the compression zone

M = resisting moment of section

M_c = moment of concrete compressive force about the neutral axis

M_{ult} = ultimate resisting moment of section

m = plastic modular ratio, $\frac{\varepsilon_u E_s}{0.85\ k_1 f_c'}$

p_{wb} = steel ratio for balanced condition at ultimate strength

q_w = tensile reinforcement index for unsymmetric T-beams

q_{wb} = tensile reinforcement index for balanced conditions at ultimate strength

S = static moment of area A about the f axis

T = total tensile force

Y = depth of point of intersection of neutral axis and rear face of an unsymmetric T-beam

α = angle of inclination of neutral axis

ε = concrete strain

ε_c = maximum concrete strain

ε_u = maximum concrete strain at ultimate strength

ε_s = steel strain

ε_y = yield strain of tensile reinforcement

GENERAL THEORY

It is convenient first to establish equations expressing the properties of the concrete "stress block," for a concrete stress distribution of general form acting on a triangular compression zone, as in Fig. A-1.

Consider a triangular compression zone, base width b_c, height c acted on by concrete stresses distributed as a function of the strain $f = F(\varepsilon)$. The strain is assumed to vary linearly from zero at the base of the triangle to a maximum ε_c at the apex.

The total compressive force C, and its moment M_c, about the neutral axis are:

$$C = \int_0^c b\, f\, dx \qquad \text{(A1)}$$

$$M_c = \int_0^c b\, f\, x\, dx \qquad \text{(A2)}$$

where b is given by

$$b = \frac{b_c}{c}\,(c - x) \qquad \text{(A3)}$$

This distance x may be expressed in terms of the strain as in Eq. (A4) below:

$$\left.\begin{aligned} x &= \frac{c}{\varepsilon_c}\,\varepsilon \\ \text{and} \qquad dx &= \frac{c}{\varepsilon_c}\,d\varepsilon \end{aligned}\right\} \qquad \text{(A4)}$$

Substituting Eq. (A3) and (A4) in Eq. (A1) leads to:

$$C = \frac{b_c\, c}{\varepsilon_c^2}\left[\varepsilon_c \int_0^{\varepsilon_c} f\, d\varepsilon - \int_0^{\varepsilon_c} f\varepsilon\, d\varepsilon\right] \qquad \text{(A5)}$$

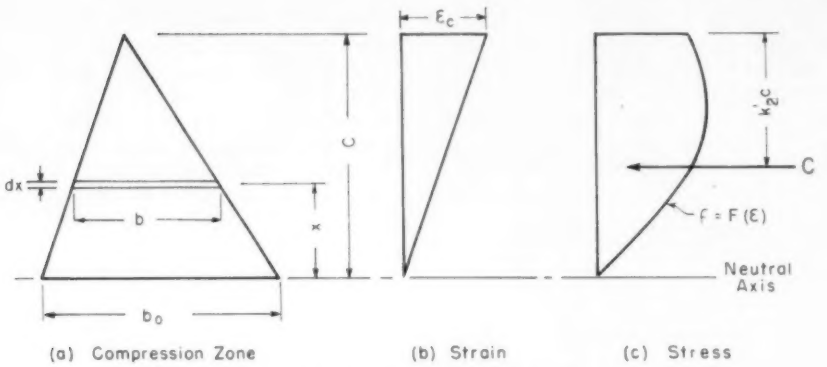

Fig. A-1—Properties of "stress block" for a triangular compression zone

It follows therefore, that the average stress on the concrete compression zone (f_{avg}) is:

$$f_{avg} = \frac{2}{\varepsilon_c^2}\left[\varepsilon_c \int_0^{\varepsilon_c} f d\varepsilon - \int_0^{\varepsilon_c} f\varepsilon d\varepsilon \right] \quad \text{(A6)}$$

Substituting Eq. (A3) and (A4) into Eq. (A2) leads to:

$$M_c = \frac{b_o c^2}{\varepsilon_c^3}\left[\varepsilon_c \int_0^{\varepsilon_c} f\varepsilon d\varepsilon - \int_0^{\varepsilon_c} f\varepsilon^2 d\varepsilon \right] \quad \text{(A7)}$$

The ratio, k_2' of the distance from the apex of the triangle to the center of action of the resultant compressive force, to the depth c of the compression zone is given by:

$$k_2' = 1 - \frac{M_c}{C_c}$$

i.e.

$$k_2' = \frac{\varepsilon_c^2 \int_0^{\varepsilon_c} f d\varepsilon - 2\varepsilon_c \int_0^{\varepsilon_c} f\varepsilon d\varepsilon + \int_0^{\varepsilon_c} f\varepsilon^2 d\varepsilon}{\varepsilon_c^2 \int_0^{\varepsilon_c} f d\varepsilon - \varepsilon_c \int_0^{\varepsilon_c} f\varepsilon d\varepsilon} \quad \text{(A8)}$$

The values of f_{avg} and k_2' can therefore be determined if the function of the stress-strain curve, $f = F(\varepsilon)$, is known.

Series T—Beams with triangular compression zones

The following assumptions are made:

1. The strain distribution across the section is linear
2. The tensile strength of the concrete may be neglected
3. The concrete stress-strain curve can be expressed as $f = F(\varepsilon)$.

Since for the beams of the test series ultimate strength was controlled by compression of the concrete, this case only will be considered here.

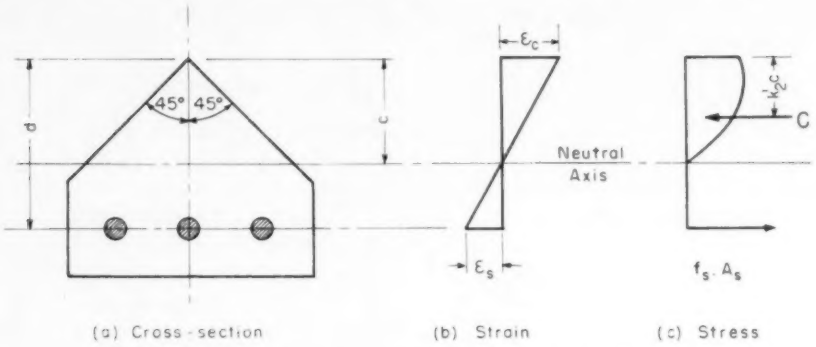

Fig. A-2—Series T: Beams with triangular compression zones

The beam cross section and the assumed strain and stress distributions are shown in Fig. A-2.

From assumption of linear strain distribution

$$\varepsilon_s = \varepsilon_c \frac{d - c}{c} \qquad \text{(A9)}$$

Tensile force

$$T = A_s f_s = A_s E_s \varepsilon_c \frac{d - c}{c} \qquad \text{(A10)}$$

Resultant compressive force $C = c^2 f_{avg}$.
From condition of equilibrium of internal forces

$$c^2 f_{avg} = A_s E_s \varepsilon_c \frac{d - c}{c}$$

which may be written:

$$c^3 + \left(\frac{A_s E_s \varepsilon_c}{f_{avg}}\right) c - \left(\frac{A_s E_s \varepsilon_c d}{f_{avg}}\right) = 0 \qquad \text{(A11)}$$

From equilibrium of internal and external moments:

$$M = A_s f_s (d - k_2' c) \qquad \text{(A12)}$$

or

$$M = c^2 f_{avg} (d - k_2' c) \qquad \text{(A12a)}$$

If the concrete stress-strain function $f = F(\varepsilon)$ is known k_2' and f_{avg} can be calculated for various values of ε_c using Eq. (A6) and (A8). These values may then be used in a trial and error solution for Eq. (A11) and (A12) to determine the maximum value of M, i.e., the ultimate moment. Alternatively, and as was actually done in this study, neutral axis depth c and moment M can be evaluated for successive increasing values of maximum concrete strain ε_c. The maximum value of M may then be picked out by inspection. This approach was greatly facilitated by the use of an LGP-30 electronic computer. The computer was also programmed to check that the steel strain ε_s was less than the yield strain of the reinforcement.

Series L—Unsymmetric T-beams

In addition to the assumptions made for the Series T beams it is also assumed that these beams are loaded vertically, and that they are free to deflect laterally. Since no moments are applied in a horizontal plane, the center of concrete compression must be vertically above the centroid of the tensile reinforcement. The distribution of stress and strain will therefore be as in Fig. A-3. The width of the stress block at the top face of the beam will be $(1/k_2')$ times the width of the web of the beam, if the reinforcement is placed symmetrically within the web. This is arrived at as follows:

PQ is the gravity axis of the compression zone, passing through P and bisecting the neutral axis TU.

Then:

$$\frac{PS}{PR} = \frac{PC}{PQ} = \frac{k_2'c}{c}$$

$$\therefore PR = \frac{PS}{k_2'} = \frac{b'}{2k_2'}$$

$$\therefore \text{Width of stress block at top face of beam} = 2PR = \frac{b'}{k_2'}$$

Ultimate strength controlled by yield of the reinforcement

From equilibrium of internal forces:

$$\frac{1}{2}\left(\frac{b'}{k_2'}\right) Y f_{avg} = A_s f_y$$

$$\therefore Y = \frac{2k_2' A_s f_y}{b' f_{avg}} \qquad \text{(A13)}$$

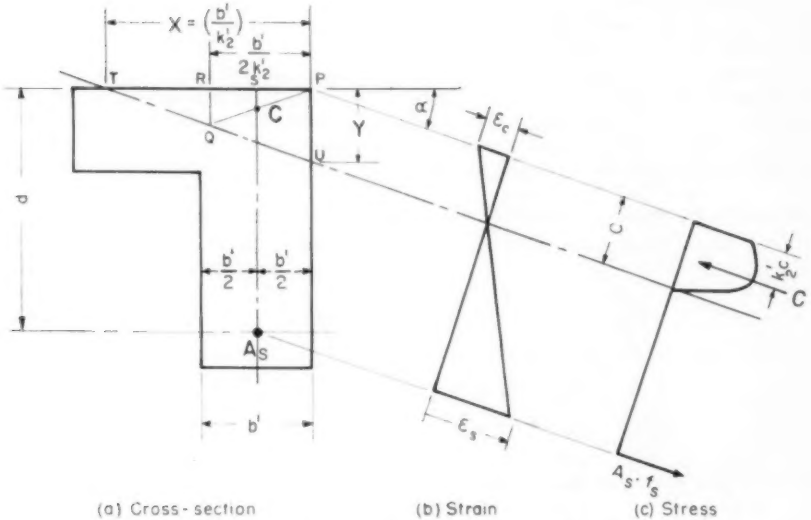

Fig. A-3—Series L: Unsymmetric T-beam

From equilibrium of internal and external moments:

$$M = \left(\frac{b'Y}{2k_2'}\right) f_{av_g} \left(d - k_2' \frac{Y}{2}\right) \quad \text{(A14)}$$

or

$$M = A_s f_y \left[d - \frac{(k_2')^2 A_s f_y}{b' f_{av_g}} \right] \quad \text{(A15)}$$

In order that the moment as given by Eq. (A15) shall be a maximum, the value of $(k_2')^2/f_{av_g}$ must be a minimum.

Differentiating this term with respect to the maximum concrete strain, ε_c, we obtain:

$$\frac{d}{d\varepsilon_c}\left(\frac{(k_2')^2}{f_{av_g}}\right) = 0 \quad \text{(A16)}$$

Substituting Eq. (A6) and (A8) into Eq. (A16) and carrying out the differentiation indicated, yields the following expression:

$$\varepsilon_c^2 A^2 - 2\varepsilon_c AS - 3ASI + 4S^2 = 0 \quad \text{(A17)}$$

Where

$$A = \int_0^{\varepsilon_c} f d\varepsilon = \text{area under stress-strain curve}$$

$$S = \int_0^{\varepsilon_c} f\varepsilon d\varepsilon = \text{static moment of area } A \text{ about the } f \text{ axis}$$

$$I = \int_0^{\varepsilon_c} f\varepsilon^2 d\varepsilon = \text{second moment of area } A \text{ about the } f \text{ axis}$$

Hence, at ultimate strength the compressive stress distribution must be such as to satisfy Eq. (A17) above. This criterion depends only on the stress-strain relationship for the concrete, $f = F(\varepsilon)$.

The strain ε_c, for which Eq. (A17) is satisfied, may be determined for concrete of various strengths. Hence the values of average concrete stress, f_{av_g}, and the ratio k_2' may be correlated with concrete cylinder strength. The appropriate values of f_{av_g} and k_2' may therefore be substituted in Eq. (A14) or (A15) in order to calculate the ultimate moment of any particular beam.

Ultimate strength controlled by crushing of the concrete

From the condition of linear strain distribution

$$\varepsilon_s = \frac{\varepsilon_c}{Y}\left[d + \left(\frac{k_2'}{2} - 1\right) Y \right] \quad \text{(A18)}$$

From equilibrium of internal forces

$$\frac{1}{2}\left(\frac{b}{k_2'}\right) Y f_{av_g} = A_s f_s$$
$$= A_s E_s \varepsilon_s \quad \text{(A19)}$$

Substituting for ε_s in Eq. (A19) and simplifying gives:

$$Y = \left(\frac{A_s E_s \varepsilon_c k_2'}{b' f_{av_g}}\right)\left[\left(\frac{k_2'}{2} - 1\right) + \sqrt{\left(\frac{k_2'}{2} - 1\right)^2 + \left(\frac{2b'd f_{av_g}}{A_s E_s \varepsilon_c k_2'}\right)}\right] \quad \text{(A20)}$$

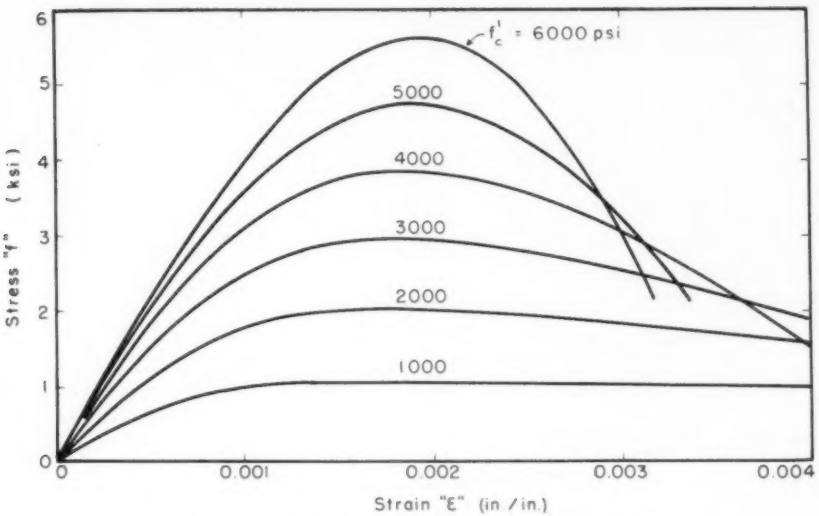

Fig. A-4—Typical concrete stress-strain curves[12]

Again, from equilibrium of internal and external moments

$$M = \left(\frac{b' Y}{2k_2'}\right) f_{avg} \left(d - k_2' \frac{Y}{2}\right) \quad \text{(A14)}$$

As in the case of the Series T beams, for any particular beam the maximum value of M may be found by trial and error by substituting the appropriate values of k_2' and f_{avg} in Eq. (A20) and (A14) corresponding to the chosen value of maximum concrete strain, ε_c. Alternatively, and as was done in this study, Y and M may be evaluated for successively increasing values of ε_c, and the maximum value of M may then be picked out by inspection. Here again the work was greatly facilitated by the use of an LGP-30 electronic computer.

In order to use the general theory it is necessary to know the concrete stress-strain function, $f = F(\varepsilon)$. The stress-strain curves used in this study were based on a statistical study by Kriz and Lee[6] of the concrete stress-strain curves obtained experimentally by Hognestad, Hanson, and McHenry.[4] The stress-strain curves are expressed as a second degree equation of the form

$$f^2 + A\varepsilon^2 + Bf\varepsilon + Cf + D\varepsilon = 0$$

in which f is the stress at strain, ε, and A, B, C, and D are coefficients which are functions of f_c', the cylinder strength. A typical series of values of the coefficients are tabulated below, and the corresponding stress-strain curves are shown in Fig. A-4.

f_c', psi	A	B	C	D
1000	−0.070	−1.421	−1.262	+1.746
2000	−0.845	−2.552	−3.291	+8.109
3000	−3.434	−3.434	−6.751	+22.58
4000	−11.51	−6.603	−14.26	+56.99
5000	−34.35	−1.196	−31.41	+137.8
6000	−56.31	+3.780	−44.13	+199.9

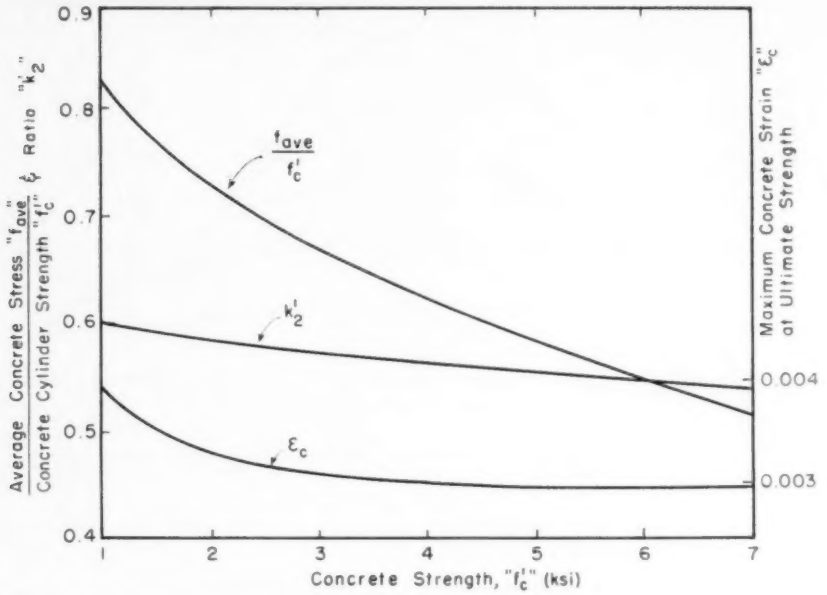

Fig. A-5—Concrete stress block properties and maximum concrete strain for ultimate strength controlled by tension in unsymmetric T-beams (Series L)

Using stress-strain curves derived in this way it was possible, using Eq. (A6) and (A8), to calculate f_{ave} and k_2' corresponding to any chosen maximum strain ε_c, for any concrete strength f_c'. These values of f_{ave} and k_2' were subsequently substituted in the appropriate equations for the ultimate strength of beams controlled by concrete compression.

For the case of the unsymmetric T-beams with ultimate strength controlled by tension it was necessary to calculate for a range of concrete strength the values of ε_c, f_{ave}, and k_2' which would satisfy the failure criterion that $(k_2')^2/f_{ave}$ must be a minimum. Curves of ε_c, f_{ave} and k_2' which satisfy this condition are plotted in Fig. A-5.

EQUIVALENT RECTANGULAR STRESS DISTRIBUTION THEORY

The following assumptions are made in order to carry out this theoretical analysis:

(1) At ultimate strength, the compressive stress in the concrete compression zone is, for purposes of computation, assumed to be uniformly distributed from the edge of maximum compressive strain over a fraction of the neutral axis depth. This fraction is 0.85 of the neutral axis depth for concrete cylinder strengths up to 4000 psi, and thereafter is reduced by 0.05 for each 1000 psi of strength in excess of 4000 psi. The uniformly distributed stress is taken to be 0.85 of the cylinder strength.

(2) Tensile strength in the concrete is neglected in flexural calculations.

(3) Plane sections normal to the neutral axis remain plane after bending, and except in the anchorage regions, strain in reinforcing bars is assumed to

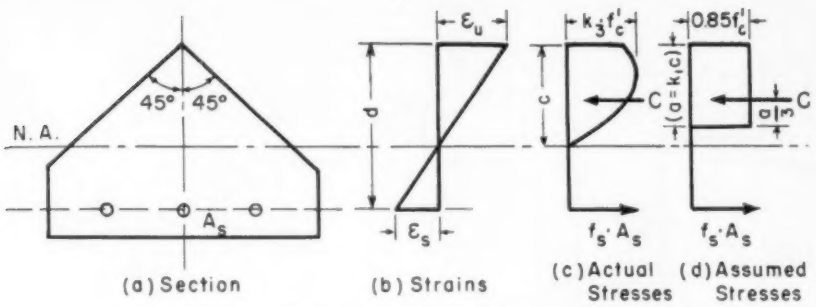

Fig. A-6—Series T: Beams with triangular compression zones

be equal to the tensile or compressive strain in the concrete at the level of the centroid of each bar or group of bars considered.

(4) The maximum strain at an extreme edge of the concrete compression zone at ultimate strength is taken as 0.003 in. per in.

(5) Stress in steel reinforcement below the minimum yield point stress for the grade of steel used is taken as 30,000,000 psi times the strain in the steel. For strain greater than that corresponding to the yield point stress, the reinforcement stress is considered independent of strain and equal to the yield point stress.

The background of the above assumptions is discussed in detail elsewhere.[8]

Series T—Beams with triangular compression zones

As in the previous analysis, only the case of ultimate strength controlled by compression will be considered.

From condition of linear distribution of strains

$$\varepsilon_s = \varepsilon_u \frac{d - c}{c} \qquad \text{(A21)}$$

From equilibrium of internal forces:

$$0.85 f_c' a^2 = A_s f_s$$

or

$$0.85 f_c' (k_1 c)^2 = A_s E_s \varepsilon_s \qquad \text{(A22)}$$

Substituting for ε_s in Eq. (A22) yields:

$$c^2 + \left(\frac{A_s E_s \varepsilon_u}{0.85 f_c' k_1^2}\right) c - \left(\frac{A_s E_s \varepsilon_u d}{0.85 f_c' k_1^2}\right) = 0 \qquad \text{(A23)}$$

From equilibrium of internal and external moments:

$$M_u = 0.85 f_c' (k_1 c)^2 (d - \tfrac{2}{3} k_1 c) \qquad \text{(A24)}$$

For any particular beam of the shape shown in Fig. A-6, the neutral axis depth c may be calculated using Eq. (A23), and hence the ultimate moment may be calculated using Eq. (A24).

Series L—Unsymmetric T-beams

The beam is considered to be loaded vertically and to be free to deflect laterally as shown in Fig. A-7. Since no moments are applied in a horizontal plane, the center of concrete compression must be vertically above the centroid of the tensile reinforcement. The width of the stress block at the top face of

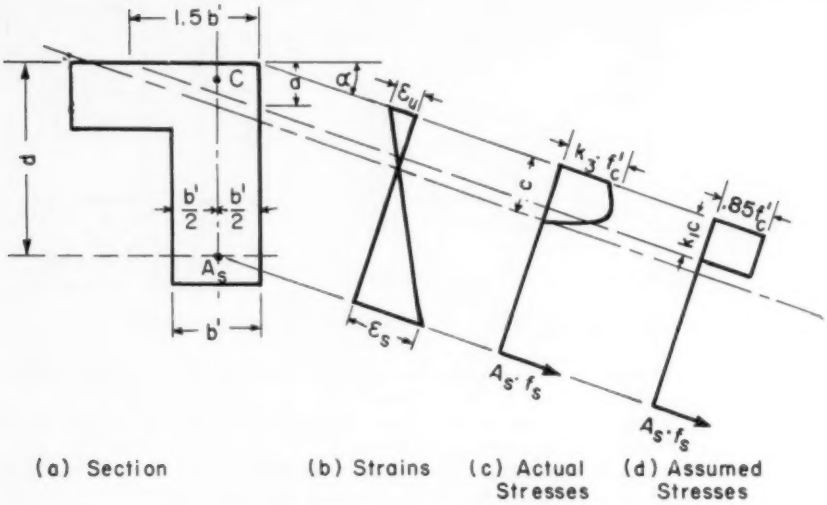

Fig. A-7—Series L: Unsymmetric T-beams

the beam will therefore be 1.5 times the width of the web of the beam for this condition to be satisfied, assuming the reinforcement is placed symmetrically within the web.

Ultimate strength controlled by yield of reinforcement — From equilibrium of internal forces:

$$\tfrac{1}{2}(1.5b'a\,0.85f_c') = A_s f_y \qquad \text{(A25)}$$

From equilibrium of internal and external moments:

$$M_{ult} = A_s f_y \left(d - \frac{a}{3}\right) \qquad \text{(A26)}$$

or

$$M_{ult} = 0.75b'a\,0.85f_c' \left(d - \frac{a}{3}\right) \qquad \text{(A26a)}$$

Solving Eq. (A25) and (A26) we have:

$$M_{ult} = A_s f_y d\,(1 - 0.52q_w) \qquad \text{(A27)}$$

or

$$M_{ult} = b'd^2 f_c' q_w\,(1 - 0.52q_w) \qquad \text{(A27a)}$$

where

$$q_w = \left(\frac{A_s}{b'd}\,\frac{f_y}{f_c'}\right)$$

Ultimate strength controlled by crushing of concrete — While the steel stress is below the yield point. From the geometry of the section:

$$c = \frac{3}{2k_1} b' \sin \alpha \qquad \text{(A28)}$$

From the assumption of linear strain distribution:

$$\frac{\varepsilon_s}{\varepsilon_u} = \frac{(d \cos \alpha + \tfrac{1}{2}b' \sin \alpha - c)}{c} \quad \text{(A29)}$$

From equilibrium of internal forces:

$$\tfrac{1}{2}(\tfrac{3}{2}b')^2 \tan \alpha \; 0.85f_c' = A_s E_s \varepsilon_s \quad \text{(A30)}$$

From equilibrium of internal and external moments:

$$M_{ult} = \tfrac{1}{2}(\tfrac{3}{2}b')^2 \tan \alpha \; 0.85f_c' (d - \tfrac{1}{2}b' \tan \alpha) \quad \text{(A31)}$$

Solving Eq. (A28), (A29), and (A30) for tan α we have

$$\tan \alpha = \left[\frac{\sqrt{\left(1 - \frac{k_1}{3}\right)^2 + \frac{3b'd}{mA_s}} - \left(1 - \frac{k_1}{3}\right)}{\frac{9(b')^2}{4k_1 mA_s}} \right] \quad \text{(A32)}$$

Where

$$m = \frac{\varepsilon_u E_s}{0.85k_1 f_c'}$$

The ultimate moment is calculated by substituting the value of tan α from Eq. (A32) in Eq. (A31).

Balanced condition — Simultaneous crushing of concrete and yield of steel. From equilibrium of internal forces:

$$\tfrac{1}{2}(\tfrac{3}{2}b')^2 \tan \alpha \; 0.85f_c' = A_s E_s \varepsilon_y \quad \text{(A33)}$$

From assumption of linear strain distribution:

$$\frac{\varepsilon_y}{\varepsilon_u} = \frac{(d \cos \alpha + \tfrac{1}{2}b' \sin \alpha) - \left(\frac{3}{2k_1} b' \sin \alpha\right)}{\left(\frac{3}{2k_1} b' \sin \alpha\right)}$$

$$\therefore \quad \frac{\varepsilon_y}{\varepsilon_u} = \frac{2k_1}{3}\left(\frac{d}{b'} \cot \alpha + \tfrac{1}{2}\right) - 1 \quad \text{(A34)}$$

Solving Eq. (A33) and (A34) for the steel ratio p_{wb} for balanced ultimate conditions:

$$p_{wb} = \frac{f_c'}{f_y}\left(\frac{1.91k_1}{3\left(\frac{\varepsilon_y}{\varepsilon_u} + 1\right) - k_1}\right) \quad \text{(A35)}$$

or

$$q_{wb} = \frac{1.91k_1}{3\left(\frac{\varepsilon_y}{\varepsilon_u} + 1\right) - k_1} \quad \text{(A35a)}$$

The ultimate moment of resistance is given by substituting q_{wb} in Eq. (A27)

$$M_{ult} = A_s f_y d \; (1 - 0.52q_{wb}) \quad \text{(A36)}$$

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Title No. 57-37

Investigation of Compressive Strength of Molded Cylinders and Drilled Cores of Concrete

By BRYANT MATHER and WILLIAM O. TYNES

This work was done to obtain information on methods being used to estimate the 28-day compressive strength of concrete. Its primary purpose was to determine the relation between the 28-day compressive strength of 6 x 12-in. cylinders molded from samples of concrete mixtures from which aggregate larger than 1½ in. had been removed, and that of 6-, 8-, and 10-in. diameter cores drilled from test structures and containing aggregate graded to 3-in. or 6-in. size. A secondary purpose of the study was to determine the strength relations of 6 x 12-in. concrete cylinders cured in the field and comparable cylinders cured in the laboratory under standard conditions.

The results suggest that estimates of the 28-day compressive strength of concrete containing 3- or 6-in. aggregate will not be significantly different whether based on results of compressive strength tests of similarly cured 6 x 12-in. cylinders of the same concrete wet-screened over a 1½-in. sieve, or on results of tests of 6-, 8-, or 10-in. diameter cores drilled from the structure and having a height equal to twice the diameter. The results suggest, however, that the smaller the core diameter, the larger the number of test cores should be to yield results of a given precision. It is indicated that cores should be more than 11 in. in diameter to give as precise an estimate of strength as that given by an equal number of 6 x 12-in. cylinders. As has been indicated by many previous studies, comparison of the field-cured and standard-cured cylinders showed that concrete cured at lower temperatures has a lower compressive strength than concrete cured under standard conditions.

■ THE COMPRESSIVE STRENGTH OF CONCRETE is the property most frequently employed as an index to the class or quality of a concrete mixture. Normally, references to the compressive strength of concrete refer to the average value obtained from results of tests of three or more 6-in. diameter, 12-in. high molded cylinders containing coarse aggregate of not more than 2 in. nominal maximum size, moist-cured at 73.4 ± 2 F for 28 days. Many factors influence the indicated strength of concrete test specimens; the American Concrete Institute standard "Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-57)" lists some of these factors. W. H. Price in 1951[1] summarized data on them.

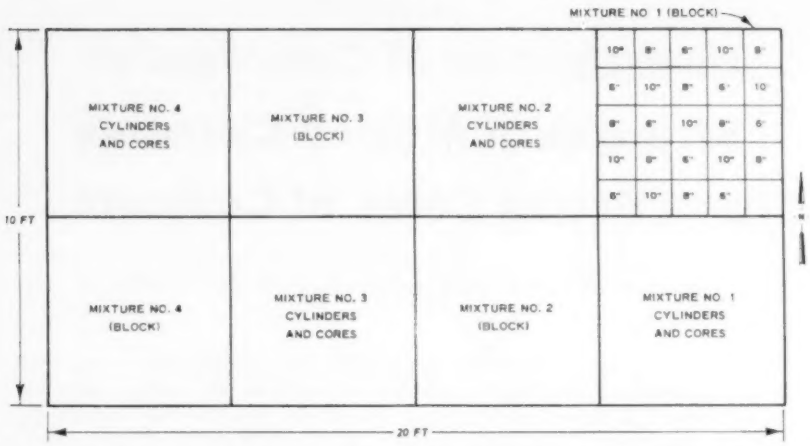

Fig. 1—Position and orientation of test blocks and curing areas and layout for drilling cores of various diameters from test block

This work was done to obtain information on methods being used to estimate the 28-day compressive strength of concrete. The effects on the compressive strength of molded cylinders and drilled cores of the following were investigated:

a. Removing aggregate larger than 1½ in. by wet-sieving mixtures containing aggregates graded up to 3 in. or 6 in.

b. Field curing.

c. Diameter of drilled concrete cores.

Four air-entrained concrete mixtures, in which aggregate size, cement content, and consequently, the water-cement ratio were varied, were used in the study. Neutralized vinsol resin was used as the air-entraining admixture. Two mixtures contained 6-in. aggregate and two 3-in.; one of each had a cement content of 2½ bags per cu yd, the other 4 bags. The water-cement ratios varied from 0.42 to 0.80. All mixtures had 5 ± 1 percent air and a slump of 1½ to 2 in. as determined on samples passing a 1½-in. sieve. All of the aggregate was crushed limestone.

From a single batch of concrete representing each of the mixtures, mixed in a 2-cu yd tilting mixer, one 20 x 60 x 60-in. test block and sixteen 6 x 12-in. cylinders were molded. Eight of the cylinders were given standard laboratory moist curing, the other eight were stored outdoors in damp sand alongside the test block for 26 days. When the blocks were 21 days old, 24 cores were drilled from each block, eight each of 6-in., 8-in., and 10-in. diameter, and stored in sand beside the block. The locations in each block from which the cores of various diameters were drilled are shown in Fig. 1. At an age of 26 days, the field-cured cylinders and the cores were brought to the laboratory.

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A member of ACI since 1944, Mr. Mather is also a member of ASTM, U.S. Committee on Large Dams, AIME, Highway Research Board committees, and other professional societies. He is also chairman of ASTM Committee C-9.

William O. Tynes (ACI member), a supervisory materials engineer (concrete research) with the U. S. Army Engineer Waterways Experiment Station is chief of the General Investigation Section, Concrete Branch, Concrete Division. Mr. Tynes has been with the Waterways Experiment Station since 1946 and has had the title of engineer (concrete research) since 1949.

The 6-in. and 8-in. cores were cut to 12-in. and 16-in. lengths, respectively, by removing 4 in. and 2 in., respectively, from each end. The field cylinders and cores were soaked in water for 44 ± 4 hr before testing. All cores and cylinders were tested for compressive strength at an age of 28 days.

TESTS OF SAMPLES OF FRESHLY MIXED CONCRETE

Samples of the freshly mixed concrete were tested for slump, air content, and bleeding. The results were:

Mixture	Aggregate size, in.	Cement content, bag per cu yd	Water-cement ratio		Slump, in.	Air, percent	Bleeding, percent
			Weight	gal. per bag			
1	6	2½	0.62	7.00	1½	4.1	3.7
2	6	4	0.42	4.74	2	5.8	1.5
3	3	2½	0.80	9.03	1¾	4.0	1.2
4	3	4	0.50	5.64	2	5.3	0.7

COMPRESSIVE STRENGTH TESTS

Results of tests of individual specimens, and averages are given in Table 2. A summary of results is given in Table 1. Each value in this table represents the average of eight tests.

DISCUSSION

The particular variables under study in this investigation do not appear to have been discussed by previous investigators. A number of related comments have been published, however, including the following:

a. Kesler and Siess[2] point out that: "Cores are often drilled from hardened concrete when the results of the standard test cylinders are unsatisfactory or when investigations are made of old structures . . . It is generally accepted that the diameter of the test specimen should be at least three times the nominal size of the coarse aggregate . . . If the aggregate is too large for the size of mold available, the oversize

TABLE 1 — SUMMARY OF COMPRESSIVE STRENGTH RESULTS

Specimen type	Diameter, in.	Approximate curing temperature, F	Mixture 2, 6-in. aggregate, *CF*,* 4 bag per cu yd, *W/C*,† 0.42 by weight			Mixture 4, 3-in. aggregate, *CF*, 4 bag per cu yd, *W/C*, 0.50 by weight			Average coefficient of variation, percent
			Compressive strength, 28 days, psi	Standard deviation, psi	Coefficient of variation, percent	Compressive strength, 28 days, psi	Standard deviation, psi	Coefficient of variation, percent	
Cylinder¶	6	73‡	4490	170	3.8	3510	98	2.8	
Cylinder¶	6	50§	3930	212	5.4	3190	128	4.0	
Core	6	50§	3970	232	5.8	3260	289	8.9	
Core	8	50§	3960	216	5.5	3310	172	5.2	
Core	10	50§	3880	136	3.5	3170	74	2.3	
			Mixture 1, 6-in. aggregate, *CF*, 2½ bag per cu yd, *W/C*, 0.62 by weight			Mixture 3, 3-in. aggregate, *CF*, 2½ bag per cu yd, *W/C*, 0.80 by weight			
Cylinder¶	6	73‡	2330	45	1.9	1870	43	2.3	2.7 } 3.0
Cylinder¶	6	50§	2220	45	2.0	1700	32	1.9	3.3 }
Core	6	50§	2080	111	5.3	1610	175	10.9	7.7 }
Core	8	50§	2130	209	9.8	1600	61	3.8	6.1 } 6.0
Core	10	50§	2130	135	6.3	1640	70	4.3	4.1 }

*CF = cement factor.
†W/C = water-cement ratio.
‡In laboratory.
§In field (mean curing temperatures: Mixture 1, 46.0; Mixture 2, 54.1; Mixture 3, 52.6; and Mixture 4, 48.0).
¶Made from concrete from which aggregate larger than the 1½-in. sieve had been removed by wet sieving.

aggregate may be removed by wet-screening. If a mold having a diameter less than three times the maximum size of the aggregate is used, the compressive strength will be lowered . . . The size of the cylinder itself affects the observed compressive strength; . . . the strength of a cylinder 36 in. in diameter by 72 in. high may be only about 82 percent of that of a standard 6 x 12-in. cylinder . . . Cores are generally taken from partially dry concrete, but the moisture content may vary considerably. To obtain a degree of uniformity in the test specimens at the time of test, cores (are required to) be soaked in water at room temperature for 40 to 48 hr immediately prior to being tested and . . . tested wet . . . The compressive strength of concrete cylinders or cores tested dry will be greater than that of comparable specimens tested wet."

b. Price[1] discussed a large number of factors that influence either the actual or indicated compressive strength of concrete test specimens.

TABLE 2 — COMPRESSIVE STRENGTH, PSI, 28 DAYS

Mixture No.	Aggregate size, in.	Cement content, bag per cu yd	W/C by weight		Cylinders		Cores		
					Standard-cured	Field-cured	6 in.	8 in.	10 in.
1	6	2½	0.62		 2300	2210	2190	2430	2060
					 2250	2230	2070	1970	2060
					 2380	2230	1880	2020	1900
					 2310	2290	1950	2000	2250
					 2290	2150	2150	2020	2040
					 2360	2230	2150	2100	2170
					 2320	2160	2100	2030	2240
					 2370	2240	2160	2500	2300
				Avg.	2330	2220	2080	2130	2130
2*	6	4	0.42		4460	4490 3940	3740	4070	3810
					4670	4460 3480	3870	4110	3700
					4670	4170 4050	3960	3750	3810
					4440	4780 4100	3850	4010	4030
					4630	4470 3870	3670	3550	3880
					4510	4560 4140	4150	4170	3730
					4720	4420 4030	4350	3890	4040
					4720	4550 3820	4150	4130	4010
				Avg.	4600	4490 3930	3970	3960	3880
3	3	2½	0.80		.....	1800 1740	1610	1610	1590
					.....	1860 1720	1590	1630	1650
					.....	1830 1720	1660	1620	1680
					.....	1850 1690	1580	1560	1750
					.....	1920 1700	1900	1550	1520
					.....	1910 1640	1410	1650	1630
					.....	1860 1690	1780	1690	1590
					.....	1910 1730	1370	1500	1680
				Avg.	1870	1700	1610	1600	1640
4*	3	4	0.50		3640	3570 3180	3530	3200	3170
					3770	3560 3180	3390	3330	3070
					3700	3410 3270	3030	3290	3290
					3640	3660 3180	3300	3560	3110
					3750	3400 3330	3100	3240	3200
					3770	3570 3250	2940	3060	3090
					3680	3540 3220	3780	3560	3230
					3520	3400 2900	3050	3260	3170
				Avg.	3680	3510 3190	3260	3310	3170

*Two additional small batches of concrete were made from Mixture 2 and Mixture 4 in a laboratory 10-S tilting mixer. From each of these batches, a series of eight 6 x 12-in. cylinders was made using concrete wet sieved over a 1½-in. sieve. The cylinders were given standard curing. The results are shown in the table. It will be noted that these averages are approximately 3 percent and 5 percent higher than those obtained from tests of similar cylinders made from concrete wet sieved from the field-mixed batches.

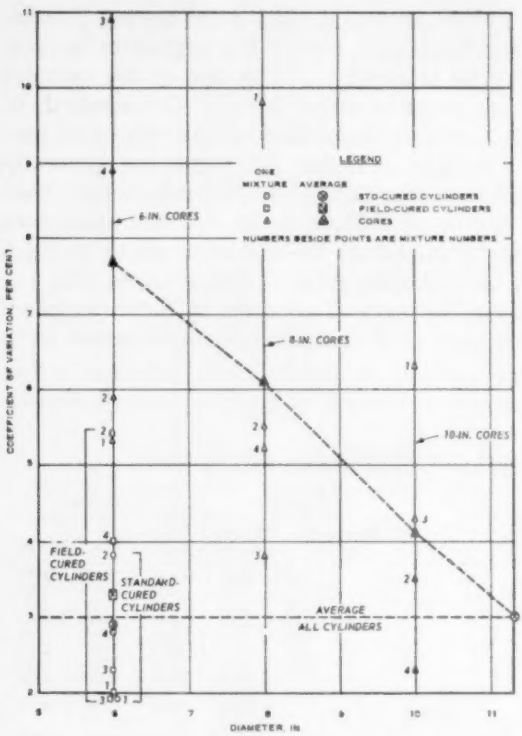

Fig. 2—Relation of coefficient of variation of test results to types and size of specimen

He points out the following: "The working stresses used in concrete design . . . are usually based on the 28-day compressive strengths of 6 x 12-in. control cylinders made from representative samples of the concrete and moist-cured at 70 F for the full 28-day period . . . Even if the structure and control cylinders were cured under identical conditions the control cylinders would not be completely indicative of the strength of the concrete in the structure because of differences in size, rate of loading, and lateral restraint . . . The indicated strength of concrete made and cured under the same conditions is different when tested in cylinders of different height and diameter . . . This difference is believed due to the possible faster strength gain of the smaller diameter cylinders. The strengths of 10-in. diameter cores and 22-in. diameter cores drilled from 5-year old concrete of Shasta and Friant Dams test blocks were practically the same, indicating that age may equalize difference due to difference in diameter of the specimen."

c. RILEM *Bulletin* 39[3] includes a report that reviews 52 papers on relations between strength of concrete as affected by type of specimen. Excerpts from this report follow:

1. "Tucker[51]* has shown . . . that homothetic† (and homogeneous) specimens give the same compressive strength but that the scattering is decreasing when the diameter is increasing if a is the ratio of the sections of the homothetic specimens A and B (or y the ratio of diameters), σ_a and σ_b are the mean standard deviations respective to each

*Numbers in brackets refer to references in RILEM *Bulletin* 39.

†Homothetic specimens are similar, are similarly placed, and have a constant ratio of similitude.

specimen we have when the number of specimens is the same for each case,

$$\sigma_a \cdot \sqrt{a} = \sigma_b$$

or

$$\sigma \cdot {}_y = \sigma_b$$

2. "Price[40] found strength to decrease with increase in diameter: 10 x 20 was 4 percent > 25 x 50; Thaulow[48] found 2.5 percent; Gonnerman[16] found 1 percent; Tucker[51] found 0 percent.

3. "Thaulow[48] and Brandtzaeg[7] believe this difference is related to quality of concrete.

4. "Price[40] thinks smaller specimens harden faster.

5. "Hamada[21] found that 25 x 50 cylinders are 10 percent < 15 x 30.

6. "Tucker[34] discussing Mather thinks it is useless to choose arbitrary correction factors. Mather proposes specifications should say how to test.

7. "A coefficient of variation of 10 percent does not seem exaggerated."

d. Tucker[4] states that in compression tests of concrete cylinders having a constant ratio of length to diameter it was found that the compressive strength of the cylinders was approximately the same regardless of the diameter, but the standard deviation of the compressive strengths decreased with increase in cylinder diameter. This was in accord with computations based on the theory of probability. Only one-fourth of the number of 12-in. diameter as of 6-in. diameter cylinders need be tested, but the weight of the 12-in. cylinder is eight times that of the 6-in.; hence twice the weight of concrete would have to be tested. "Thus, if we reduce the diameter of the test specimen, we need only make the number proportional to the ratio of the areas to obtain equal statistical information."

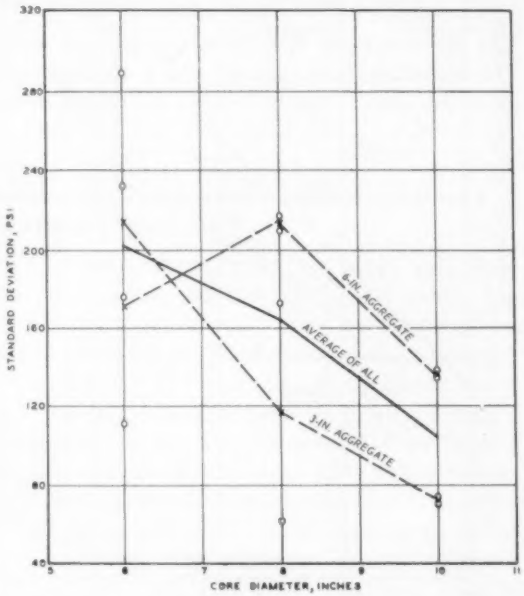

Fig. 3—Relation of standard deviation of test results to diameter of core and size of aggregate

VARIANCE OF RESULTS

The standard deviation and coefficient of variation for each group of eight tests and for all specimens of any one type given in Table 1 are shown graphically in Fig. 2 and 3 and tend to confirm the findings of Tucker as quoted by RILEM that scatter decreases as diameter increases. Fig. 2 suggests that cores 11.3 in. in diameter would indicate compressive strength as precisely as an equal number of molded 6 x 12-in. cylinders. Fig. 3 shows the trend of the standard deviation to vary inversely with core diameter. The correlation coefficient for the line representing the average standard deviations of the specimens

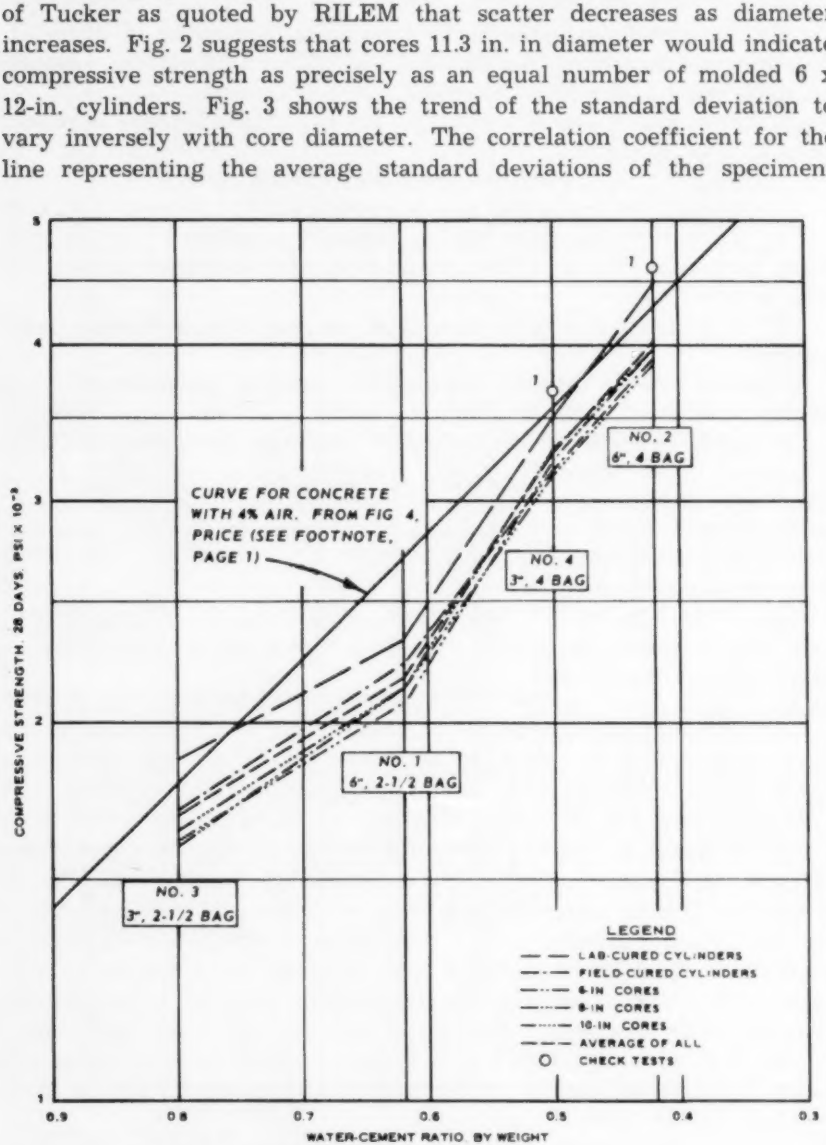

Fig. 4—Compressive strength–water-cement ratio relation

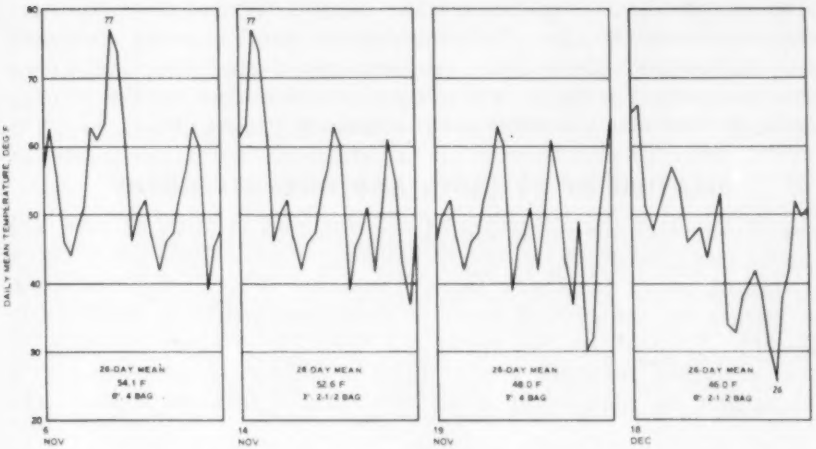

Fig. 5—Mean daily temperature during 26-day field-curing period

from the two mixtures with 3-in. aggregate was 0.742; that of the line representing the average standard deviations of the specimens from all four mixtures was 0.523.

RELATION OF OBSERVED RESULTS TO OTHER DATA

Fig. 4 illustrates the relation between the average strengths of the five types of specimens tested in this study to the average curve for the relation of compressive strength to water-cement ratio of air-entrained concrete given by Price.[1]

RELATION OF FIELD-CURED AND STANDARD-CURED CYLINDER STRENGTHS

Coefficients of variation of the strengths of groups of eight cylinders averaged 3.0 percent (2.7 for standard cured, 3.3 for field cured); for the four mixtures, the standard-cured cylinder strength groups showed coefficients of variation ranging from 1.9 to 3.8 percent while the range for the field-cured cylinders was 1.9 to 5.4 percent. It was concluded that these variances were sufficiently homogeneous to justify a study of significance by the analysis of variance. The calculated F ratio was 99.61, which greatly exceeds the values of 4.03 and 7.17, respectively, that are required to be significant at the 5 percent and the 1 percent level, respectively. It may, therefore, be concluded that the separation of the curve for the standard-cured cylinder strengths from the curve for the field-cured cylinder strengths indicated in Fig. 4 does in fact indicate a very significant difference. This relation is that which was expected. Test blocks and cylinders were made in the winter, and the

blocks and field-cured cylinders were cured at temperatures substantially lower than the 73.4 ± 2 F at which the standard-cured specimens were maintained. Mean daily temperatures during the field-curing period are plotted in Fig. 5. Mean temperatures during the field-curing periods for the four concretes ranged from 46.0 to 54.1 F.

RELATION OF CYLINDER AND CORE STRENGTHS

Fig. 6 indicates the departure of the observed relation of core and cylinder strengths from the line of equality. The analysis given under variance of results has shown that the variance of compressive-strength test results for the several types of specimens tested is significantly different. Youden[5] (p. 56) points out that: "In a study of three or more analytical methods which differ markedly in precision, it is obvious that the comparison of the more precise methods is associated

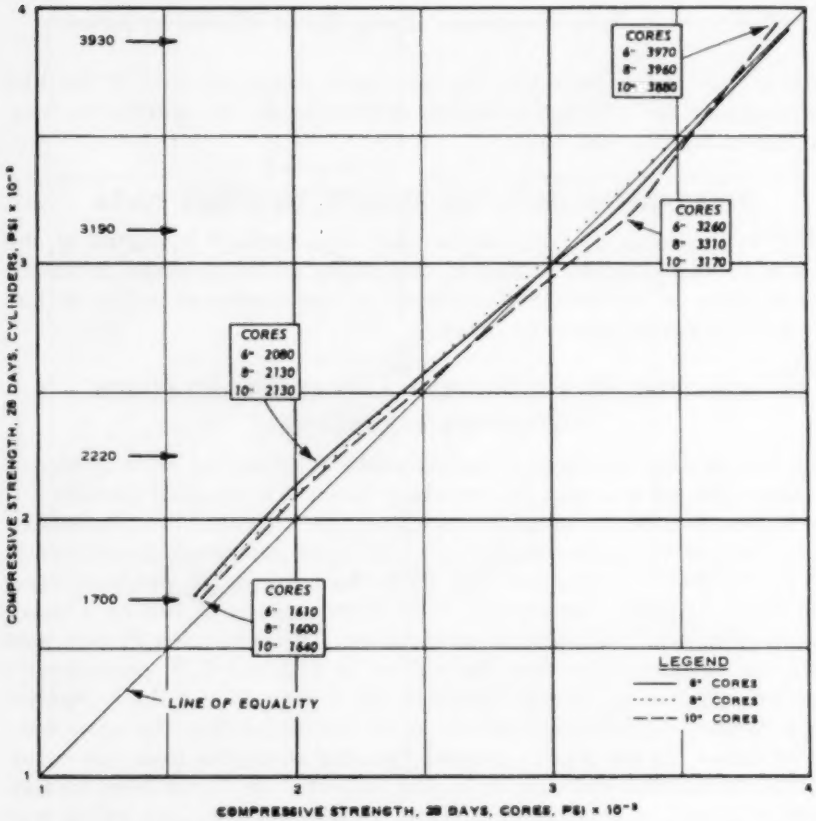

Fig. 6—Relation of cylinder strength to core strength, 50 F curing

with a smaller variance than comparisons involving data with less precision. There is, in this case, no possibility of postulating a variance that holds for all the measurements, and the estimation of variance . . . is without physical meaning. A kind of pooled estimate results if the arithmetic is performed, but this estimate is not strictly applicable to any particular pair of methods." In view of this situation, the conclusion that there appeared to be no significant difference in the indicated compressive strength for any of the four concretes studied, depending on whether the strength was estimated by tests of 6 x 12-in. cylinders, 6 x 12-in. cores, 8 x 16-in. cores, or 10 x 20-in. cores provided all received similar curing, regardless of whether the concrete contained 3-in. or 6-in. aggregate, was drawn primarily from an inspection of the test results themselves (Table 2), from the lack of separation of the lines for the four types of testing in Fig. 4, and from the lack of separation of the three lines in Fig. 6 from the line of equality.

SUMMARY OF RESULTS AND CONCLUSIONS

Test specimens representing four concrete mixtures were tested for compressive strength at an age of 28 days. Five groups of specimens were tested to represent each mixture. The five groups included four types of cylindrical specimens, each having a height equal to twice the diameter; for one type of specimen, two groups were made and cured under different conditions. Each group included eight specimens. Average strengths and coefficients of variation are given in Table 1.

These results and the analyses that have been made of them suggest the following relations:

a. Concrete cured at approximately 50 F has a significantly lower 28-day compressive strength than concrete cured at 73.4 ± 2 F.

b. Concrete cured at approximately 50 F has essentially the same indicated compressive strength regardless of which of the four types of test specimens are used to measure it.

c. No significant differences were found between the degree to which the four types of specimens served to indicate the compressive strength of the various mixtures.

d. Molded 6 x 12-in. cylinders made from concrete from which aggregate larger than the 1½-in. sieve had been removed by wet sieving gave the most precise indication of 28-day compressive strength.

e. Diamond-drilled cores gave indications of 28-day compressive strength with a precision that increased with increasing core diameter from 6 in. to 8 in. to 10 in.

These relations suggest the following general conclusions concerning the indication of the level of 28-day compressive strength of similarly cured concrete that is obtained from tests of cylindrical specimens having a height equal to twice the diameter, taken from concrete mixtures containing either 3-in. or 6-in. aggregate and water-cement ratios in the range 0.4 to 0.8 by weight, when the specimen type selected is one of the four investigated in this study.

a. The accuracy with which the strength level is indicated will not be significantly affected by the type of specimen.

b. The precision with which the strength level is indicated will be significantly affected by the type of specimen.

1. Molded 6 x 12-in. cylinders made from concrete wet sieved over the 1½-in. sieve give results of a given precision with fewer specimens than are given by the same number of diamond-drilled cores of either 6-in., 8-in., or 10-in. diameter.

2. The precision of the results given by diamond-drilled cores increases with increasing core diameter. The results suggest that for a diameter of slightly more than 11 in., the precision of results from core tests would be equal to that of wet-sieved, 6 x 12-in. cylinder tests.

ACKNOWLEDGMENTS

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Title No. 57-38

Freezing and Thawing Tests of Lightweight Aggregate Concrete

By PAUL KLIEGER and J. A. HANSON

Nine lightweight aggregates and one natural sand and gravel aggregate were used in concretes subjected to laboratory freezing and thawing tests and, in some cases, to tests for resistance to deicer scaling. Concretes were prepared at two levels of compressive strength: 3000 psi and 4500 psi at 28 days. Both non-air-entrained and air-entrained concretes were prepared, using the aggregates in an air-dried condition and in a 24-hr saturated condition.

The results of these tests indicate the necessity for providing intentionally entrained air to attain a high level of durability, the importance of moisture content of aggregate, and the influence of strength level, i.e., water-cement ratio, on the durability. The results point to the desirability of evaluating a lightweight aggregate by means of laboratory freezing and thawing tests of air-entrained concrete made with the aggregate, as is generally done for normal weight aggregate.

■ THE USE OF LIGHTWEIGHT AGGREGATE in concrete is increasing rapidly, particularly in structural applications. Practical field experiences and comprehensive laboratory investigation* have shown that high quality concrete can be produced with this type of aggregate.

Exposed concrete is subject to the potential deteriorating influences of weather and other aspects of environment. The resistance of concrete to freezing and thawing is of particular importance when considering its use in exposed applications. Concretes made with normal weight aggregates have been shown to have excellent resistance to this type of weathering when they contain entrained air supplied by an acceptable air-entraining material. However, the level of durability attained is influenced also by the properties of the normal weight aggregates.

There appears to be little information available as to the resistance of lightweight aggregate concrete to freezing and thawing. The technique of providing protection by the use of entrained air should be just as valid and as necessary for lightweight aggregates as for normal weight aggregates. This study was undertaken to provide comprehensive laboratory data on the resistance to freezing and thawing of concretes

*Shideler, J. J., "Lightweight Aggregate Concrete for Structural Use," ACI JOURNAL, V. 29, No. 4, Oct. 1957 (*Proceedings* V. 54), pp. 299-328.

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made with and without entrained air and a number of different lightweight aggregates.

SCOPE OF TESTS

Nine lightweight aggregates and one normal weight sand and gravel aggregate were used in this study. Concretes were prepared at two levels of compressive strength: 3000 psi and 4500 psi at 28 days of age. For each strength level, both non-air-entrained and air-entrained concretes were prepared using the aggregates in two different moisture conditions: (1) dried in the air of the laboratory to a moisture content about 0.1 to 0.2 percent by weight, and (2) after water soaking 18-24 hr.

For the air-entrained concretes, an air-entraining agent was added at the mixer to provide 3 to 4 percent more air than that of the non-air-entrained mixes.

Concrete prisms were made for the freezing and thawing tests. Some cylinders were prepared for compressive strength, unit weight, and absorption determinations. Three of the lightweight aggregates were also used in 4500-psi air-entrained concrete slabs for the determination of the resistance of slab surfaces to scaling resulting from the use of calcium chloride as a deicer.

Materials

The cement used in making the concrete prisms was a blend of four Type I portland cements purchased in the Chicago area. Concrete for the scaling slabs was made using a later blend of these same four brands.

The nine lightweight aggregates and the one normal weight aggregate (No. 8) included in this study are briefly described as follows:

Aggregate 2 — An expanded shale produced in a rotary kiln. The raw material is screened only for maximum size prior to bloating. The coarser sizes are rounded and have a smooth shell. The fine material, passing the No. 4 sieve, is obtained by crushing the coarser material.

Aggregate 3 — An expanded shale produced in a rotary kiln. The raw material is crushed and screened to four sizes. These individual fractions

TABLE 1 — GRADING OF AGGREGATES

Sieve size	Percentage passing sieve size indicated, by weight										
	Lightweight aggregates									Normal weight aggregate	
	No. 2, 3, 4, 5, 6, and 7		No. 10			No. 11		No. 12		Elgin No. 8	
	Fine	Coarse	Fine	Medium	Coarse	Fine	Coarse	Fine	Coarse	Fine	Coarse
¾ in.	100	100	100	100	100	100	100	100	100	100	100
⅜ in.	100	50	100	100	21	100	27	100	41	100	50
No. 4	100	0	100	83	1	100	2	100	2	100	0
No. 8	85	0	100	19	0	85	0	76	0	82	0
No. 16	60	0	78	2	0	50	0	45	0	67	0
No. 30	40	0	49	0	0	28	0	25	0	43	0
No. 50	22	0	25	0	0	16	0	14	0	13	0
No. 100	10	0	9	0	0	10	0	9	0	5	0

are passed through separate kilns. All particles are rounded and have a smooth shell down to and including material retained on the No. 100 sieve.

Aggregate 4 — An expanded clay produced in a rotary kiln. The raw material is passed through a 1-in. screen and into the kiln. Most of the bloated material from the kiln does not require crushing except to produce additional material passing the No. 50 sieve. The bloated material is very fine grained, but the individual particles are not rounded and they do not have a shell.

Aggregate 5 — An expanded slate produced in a rotary kiln. The coarser particles are angular and porous, but are not as sharp as Aggregates 6 and 7. Material passing the No. 4 sieve is obtained by crushing.

Aggregate 6 — An expanded slag produced by spraying a controlled amount of water on molten slag. Most of the sizes are obtained by crushing; all particles are very angular and have a porous surface texture.

Aggregate 7 — Produced by burning on a sintering grate a carbonaceous shale from anthracite coal processing. All sizes of aggregate are obtained by crushing. Individual particles are very sharp and angular, have a porous surface texture.

Aggregate 8 — Elgin sand and gravel, a natural uncrushed material. The gravel is well rounded, and is about half calcareous and half siliceous material. The sand is predominately quartz. The aggregate has a good service record in northern Illinois.

Aggregate 10 — Made from shale expanded in a rotary kiln. The material was separated into several size fractions prior to bloating and the finished product is generally rounded and sealed.

Aggregates 11 and 12 — Expanded shales produced in a rotary kiln. The raw material is screened only for maximum size prior to bloating. The coarser sizes are rounded and have a smooth shell. The fine material, passing the No. 4 sieve, is obtained by crushing the coarser material.

The total of ten aggregates included five expanded shales, an expanded slate, an expanded clay, a shale expanded by the sintering process, an expanded blast-furnace slag, and the normal weight sand and gravel.

The gradings of the aggregates as used in these tests are shown in Table 1. All aggregates except Aggregates 10, 11, and 12 were air dried and screened into the various size fractions. The fine aggregate size fractions for Aggregates 2, 3, 4, 5, 6, and 7 were recombined to provide a grading about midway between the limits established in ASTM C 330-53T for lightweight aggregates. The coarse fractions were re-

TABLE 2 — ABSORPTION, SPECIFIC GRAVITY, AND UNIT WEIGHT OF AGGREGATES

Aggregate No.	Size	Absorption, percent by weight*			Specific gravity*			Unit weight,† lb per cu ft	
					Bulk		Apparent		
		1 hr	4 hr	24 hr	Oven-dry	S.S.D.		Dry loose	Dry rodded
2	Fine	2.2	4.1	6.8	1.75	1.86	1.98	52.0	56.2
	Coarse	11.3	12.0	14.9	1.17	1.34	1.41	42.1	45.0
3	Fine	3.1	3.6	4.8	1.91	2.00	2.10	72.0	77.2
	Coarse	6.6	8.0	9.7	1.21	1.33	1.37	43.7	45.4
4	Fine	9.6	11.1	13.5	1.76	2.00	2.34	57.8	63.7
	Coarse	11.3	12.7	15.6	1.50	1.73	1.96	49.6	54.2
5	Fine	3.4	4.4	6.6	1.97	2.10	2.26	62.7	68.6
	Coarse	8.1	9.2	11.0	1.30	1.44	1.54	43.9	47.3
6	Fine	2.3	2.9	4.0	2.10	2.19	2.30	64.2	70.2
	Coarse	6.2	6.6	7.1	1.46	1.56	1.62	44.2	47.3
7	Fine	5.6	6.3	7.0	1.91	2.05	2.21	64.0	68.9
	Coarse	6.5	7.0	7.3	1.59	1.72	1.82	45.9	48.0
8	Fine	—	—	2.2	—	2.65	—	108.2	112.7
	Coarse	—	—	2.1	—	2.65	—	100.0	108.2
10	Fine	5.1	7.1	9.5	1.72	1.88	2.06	62.5	65.3
	Medium	7.4	9.6	11.8	1.37	1.53	1.63	50.2	53.6
	Coarse	7.7	9.4	12.3	1.28	1.43	1.51	43.8	47.0
11	Fine	7.9	10.0	11.7	1.61	1.80	1.98	58.2	65.2
	Coarse	11.8	12.7	15.6	1.44	1.66	1.85	47.1	51.8
12	Fine	8.8	10.1	11.8	1.57	1.76	1.93	53.4	62.8
	Coarse	5.4	6.7	7.7	1.32	1.42	1.47	44.1	49.0

*ASTM C 127-42.
†ASTM C 29-42 and 55T.

combined using equal weights of No. 4 to 3/8 in. and 3/8 in. to 3/4 in. Aggregates 10, 11, and 12 were used in the grading in which they were supplied by the manufacturer. The normal weight sand and gravel grading met the requirements of ASTM C 33 for concrete aggregates.

Table 2 shows the absorptions, specific gravity, and unit weights of these aggregates. Details as to the testing techniques used appear in the Shideler report.*

Neutralized Vinsol resin was used as the air-entraining admixture.

Concretes

Concrete batches were mixed in a 5/8-cu ft tub type Lancaster mixer. Aggregates were used in both the air-dried and soaked condition. When air-dried aggregate was used, the aggregate and about two-thirds of the mixing water were mixed 2 min, the cement and remaining water containing the air-entraining agent were then added and mixing continued for 3 min more. For the soaked aggregate, the aggregate containing essentially all the net mixing water was mixed for 2 min; the cement, air-entraining agent, and water required for slump adjustment were then added and mixing continued for 3 min more.

Concrete slumps were generally held in the range of 2 to 3 in. Air content was determined by a volumetric technique in accordance with the provisions of ASTM C 173-58, with the exception that the concrete

*Op. cit.

in the bowl was compacted by external vibration instead of hand rodding. Vibration insured more uniformly complete compaction of the widely different concrete mixtures included in this study. External vibration, which was supplied by means of a vibrating table, was used also to compact all test specimens. The specimens for the freezing and

TABLE 3 — CONCRETE DATA: 3000-PSI CONCRETES

Aggregate		Quantities per cu yd						Slump, in.	Air content, percent	Unit weight of plastic concrete, lb per cu ft	28-day compressive strength,‡ psi
		Cement		Gross water, lb	Aggregate†		Air-entraining agent, ml				
No.	Condition*	Sack	lb		Fine lb	Coarse, lb					
Non-air-entrained											
2	D	6.39	601	493	818	673	0	2.3	4.5	95.8	3200
	W	6.40	602	591	820	675	0	2.3	2.7	99.5	2980
3	D	5.28	496	361	996	782	0	2.5	4.0	97.7	3330
	W	5.30	498	433	1000	784	0	2.8	3.8	101.0	2710
4	D	4.54	425	556	1077	759	0	2.9	2.8	104.2	2840
	W	4.55	427	665	1082	763	0	2.4	2.0	109.2	2830
5	D	6.36	598	461	1183	654	0	2.5	3.5	107.1	3230
	W	6.21	584	540	1159	639	0	2.1	3.8	108.2	3030
6	D	6.99	657	462	1283	704	0	2.7	3.0	115.0	2860
	W	6.92	651	505	1268	696	0	2.4	2.3	115.3	2500
7	D	6.51	612	514	1232	716	0	2.7	2.0	113.8	2650
	W	6.51	612	545	1233	716	0	2.3	1.5	115.0	2730
10	D	4.53	426	390	921	707	0	2.8	4.0	90.6	2460
	W	4.42	416	504	920	702	0	2.0	3.6	94.1	1760
11	D	5.51	518	527	1105	639	0	2.0	2.8	103.2	3350
	W	5.53	520	619	1113	644	0	2.3	2.0	107.4	3930
12	D	4.97	467	417	1108	601	0	2.0	3.5	96.1	3300
	W	5.06	476	514	1129	612	0	2.5	1.5	101.1	2630
8	D	3.97	373	339	1628	1763	0	2.2	1.5	152.0	2740
	W	3.97	373	339	1626	1761	0	2.0	1.7	151.8	2910
Air-entrained											
2	D	6.16	579	445	766	630	2660	3.3	8.5	89.6	2030
	W	6.46	607	549	783	645	1810	2.4	6.5	95.6	2740
3	D	5.35	503	343	950	755	1588	2.3	7.0	94.2	3190
	W	5.31	499	392	944	740	2085	2.8	8.0	95.5	2690
4	D	4.62	434	498	1042	734	1380	2.3	7.0	100.1	3020
	W	4.64	436	630	1047	738	1255	2.6	5.7	105.4	2600
5	D	6.23	586	421	1108	611	1510	2.6	7.7	101.1	2800
	W	6.21	584	510	1137	610	1635	2.7	7.7	103.4	2510
6	D	6.95	654	418	1213	666	2435	2.9	6.6	109.6	2550
	W	7.20	677	446	1250	686	2200	2.2	6.0	113.2	2780
7	D	6.39	601	450	1150	668	4000	4.4	7.0	106.2	1770
	W	6.64	624	480	1198	693	3115	2.1	5.0	110.9	2660
10	D	4.49	422	351	888	679	990	2.6	8.0	86.8	2500
	W	4.52	425	469	890	679	865	2.8	6.6	91.1	1990
11	D	5.45	512	459	1082	625	1245	2.2	6.7	99.3	2660
	W	5.55	522	567	1093	632	1135	2.0	4.8	104.3	3320
12	D	4.97	467	395	1073	582	810	2.4	6.6	93.2	2440
	W	5.11	480	481	1102	597	710	2.5	4.4	98.5	2500
8	D	3.91	368	310	1570	1701	1250	2.8	4.5	146.3	2810
	W	3.91	368	310	1570	1700	1455	2.4	5.7	146.2	2590

*D = air dried. W = soaked 18-24 hr.
†Air-dry weights.
‡Curing: 28 days moist. One 6 x 12-in. cylinder only.

TABLE 4 — CONCRETE DATA: 4500-PSI CONCRETES

Aggregate		Quantities per cu yd						Slump, in.	Air content, percent	Unit weight plastic concrete, lb per cu ft	Cylinders‡	
		Cement		Gross water, lb	Aggregate†		Air-entraining agent, ml				Absorption, percent by weight	28-day compressive strength, psi
No.	Condition*	sack	lb		Fine, lb	Coarse, lb						
Non-air-entrained												
1	D	8.19	770	468	696	761	0	2.0	3.0	97.6	17.0	4890
	W	8.12	763	554	692	697	0	2.7	2.5	100.2	19.4	4720
3	D	7.54	709	371	861	825	0	2.2	2.6	102.4	14.1	5200
	W	7.24	681	420	818	783	0	2.7	2.2	100.1	16.3	4926
4	D	6.22	585	499	938	814	0	2.1	3.1	105.0	18.6	5630
	W	6.02	566	648	945	820	0	2.9	1.5	110.3	22.3	4920
5	D	7.74	728	439	1022	691	0	2.6	2.8	106.7	15.0	5320
	W	7.94	746	535	1019	689	0	2.5	2.1	110.7	16.3	4760
6	D	8.65	813	438	1100	736	0	3.0	2.6	114.4	16.9	4740
	W	8.79	826	465	1103	739	0	2.8	2.7	116.0	17.4	5040
7	D	8.23	773	436	1075	766	0	2.6	1.5	115.0	18.2	4630
	W	8.19	770	512	1066	760	0	2.6	1.6	115.1	18.7	4550
10	D	6.79	638	379	661	949	0	2.8	1.7	97.3	13.9	6510
	W	7.00	658	494	663	959	0	2.9	1.5	102.9	16.9	6200
10X§	D	5.86	551	331	671	975	0	2.0	2.3	95.9	13.2	6100
	W	5.73	543	503	630	975	0	2.3	1.6	100.2	16.4	4940
11	W	7.15	672	615	1006	678	0	2.6	2.1	107.9	20.7	4350
12	W	5.99	563	510	999	620	0	2.6	2.8	99.7	17.3	4150
8	D	4.90	461	239	1330	2031	0	3.0	1.0	152.2	11.9	5230
	W	4.95	465	247	1339	2095	0	2.4	0.7	153.6	11.8	5070
Air-entrained												
2	D	8.05	757	438	644	648	2614	2.7	7.5	92.1	16.2	4550
	W	7.97	749	525	651	656	2300	2.4	6.5	95.6	18.0	4450
3	D	7.27	683	348	791	758	3706	2.4	5.7	95.5	13.6	4630
	W	7.12	669	398	770	738	3607	2.8	6.0	95.4	15.5	3940
4	D	5.87	552	475	903	789	1536	2.5	6.5	100.9	17.9	4390
	W	5.85	550	618	895	777	1513	3.1	5.4	105.2	20.9	4260
5	D	7.78	731	412	974	659	1862	2.8	6.1	102.8	14.0	4630
	W	8.07	759	507	986	667	1885	2.7	5.5	103.1	16.5	4780
6	D	8.77	824	403	1055	706	1644	2.5	6.5	110.6	15.2	4710
	W	8.63	816	427	1039	729	1648	2.1	5.7	113.4	15.3	4760
7	D	8.03	766	439	1032	736	2033	2.0	5.2	110.0	16.6	4540
	W	8.13	764	475	1029	734	2147	2.3	4.9	111.2	17.0	4460
10	D	7.00	658	355	639	918	1420	2.3	5.2	95.2	12.7	6280
	W	7.13	670	464	635	912	1411	2.7	4.6	99.3	16.3	6030
10X§	D	5.81	546	352	657	942	951	2.2	5.3	92.5	12.1	5330
	W	5.79	544	467	650	932	1058	2.9	5.4	96.0	15.7	4540
11	W	7.01	659	564	941	592	1676	2.7	7.1	102.1	20.1	4500
12	W	5.97	561	467	957	594	1047	2.4	6.5	95.5	16.0	4040
8	D	4.83	454	202	1279	2001	1560	2.6	5.0	145.8	10.7	4600
	W	4.85	456	220	1280	2003	1561	2.5	4.6	146.6	12.4	4750

*D = air dried. W = soaked 18-24 hr.
†Air-dry weights.
‡Curing: 7 days moist plus 21 days in air of laboratory. Average of two cylinders.
§Repeat tests of aggregate 10, using a lower cement content.

thawing tests were 3 x 3 x 11¼-in. concrete prisms equipped with stainless steel gage studs for length change measurements. Slabs for the surface scaling tests were 3 x 6 x 15 in., the 6 x 15 in. top surface finished with a wood float after compaction. Cylinders for strength test were 6 x 12 in. All molds were steel and were watertight.

Tables 3 and 4 show the data for all of the concrete mixtures. The data in the Shideler report* served as a basis for selecting the mix proportions to provide the desired strength levels. Although strength was of secondary importance in this study, a few cylinders were made for strength tests. In the 3000-psi series, one cylinder from each concrete mixture was cured 28 days at 73 F and 100 percent relative humidity (R.H.). This was hardly sufficient to adequately evaluate the strength producing properties of these concretes, but the average strength of the non-air-entrained concretes was 2910 psi, and of the air-entrained concretes 2600 psi. For the 4500-psi concretes, two cylinders for each concrete were cured 7 days at 73 F and 100 percent R.H. followed by 21 days in air at 73 F and 50 percent R.H. prior to testing. The average strength of the non-air-entrained concretes in this series was 5170 psi, and of the air-entrained concretes 4750 psi. In terms of individual strengths, the range for both series is 1760 psi to 6510 psi.

Durability tests

Prisms and slabs were cured 14 days in the moistroom at 73 F and 100 percent R.H. followed by 14 days in the air of the laboratory at 73 F and 50 percent R. H. The prisms were then immersed in water for 3 days prior to the start of freezing and thawing tests. The surfaces of the slabs were covered with water ¼ in. in depth for 3 days prior to the start of surface scaling tests.

The prisms were frozen and thawed while immersed in tap water at all times. Two cycles of freezing and thawing were obtained every 24 hr, 7 days per week. The minimum specimen temperature attained was approximately −10 F and the maximum was approximately +55 F. In a recent study of freezing and thawing test procedures, this test procedure produced results comparable to those obtained by the procedures outlined in ASTM C 290-57T, "Test for Resistance of Concrete Specimens to Rapid Freezing and Thawing in Water (Tentative)." The test prisms were measured periodically for changes in fundamental transverse frequency (ASTM C 215), length (ASTM C 157), and weight.

In the surface scaling tests which were conducted on concrete slabs made with three of the lightweight aggregates, the slabs with ¼ in. layer of water on the surface (equal to 250 ml) were placed in a cold room maintained at 0 F overnight. The slabs were removed the following morning and stored for 6 hr in a room maintained at ap-

*Op. cit.

*D = air dried. W = soaked 18-24 hr.
†Air-dry weights.
‡Curing: 7 days moist plus 21 days in air of laboratory. Average of two cylinders.
§Repeat tests of aggregate 10, using a lower cement content.

proximately 70 F. Flake calcium chloride (78 percent $CaCl_2$) was applied directly to the ice on the surface at the rate of 2.4 lb per sq yd of surface area. At the end of the thaw period, the solution of calcium chloride and water was poured off the surface, the surface was rinsed with fresh water, and the ¼ in. layer of water added preparatory to repeating the daily cycle. Surface scaling was evaluated by visual examination and numerical ratings assigned periodically, taking into consideration both extent and depth of scaling. These numerical ratings and their description are as follows:

Rating	*Condition of surface*
0	No scaling
1	Very slight scaling
2	Slight to moderate scaling
3	Moderate scaling
4	Moderate to bad scaling
5	Severe scaling

DISCUSSION OF DURABILITY DATA

A number of different measures are available for use as criteria for assessing the durability of laboratory test specimens. Changes in weight, dynamic modulus of elasticity, and length, particularly the latter two, are in general use. Changes in dynamic modulus were used to calculate durability factors as defined in the current tentative ASTM methods for conducting freezing and thawing tests, one of which is ASTM C 290-57T.

Tables 5 and 6 show the expansions, changes in dynamic modulus, and weight changes after 300 cycles of test for all of the concretes included in this study. In addition, the individual durability factors are shown, together with the relative durability factors (relative to the durability factor for the concrete made with the normal weight sand and gravel). The individual durability factors are calculated using the number of cycles at which the relative dynamic modulus reaches 60 percent or 300 cycles if the relative dynamic modulus has not reached 60 percent at that time. Most of the discussion to follow will be based on the durability factors calculated from changes in dynamic modulus. This criterion is as reliable an indicator of distress due to freezing and thawing as is expansion during the test. On the other hand, the relationships between expansion and weight change indicate weight change to be a relatively poor indicator of damage. For example, many of the concretes showing marked reduction in dynamic modulus show gains in weight rather than losses.

Influence of air entrainment on durability

Fig. 1 and 2 show bar graphs of the individual durability factors for the concretes in this study. These durability factors are based on the changes in fundamental transverse frequency of vibration after 300 cycles of test.

In the case of the air-dried aggregates, Aggregates 2, 3, 4, and possibly 5 and 11, show only minor changes in durability factor with entrained air. The remainder of the aggregates, including the normal weight sand and gravel, show substantial increases in durability factor. For

TABLE 5 — RESULTS OF FREEZING AND THAWING TESTS: 3000-PSI CONCRETES

Aggregate No.	Condition*	Air content, percent	Expansion at 300 cycles, percent	Percent of original E at 300 cycles	Change in weight at 300 cycles, percent	Durability factor at 300 cycles	Relative durability factor at 300 cycles†
			Non-air-entrained				
2	D	4.5	0.035	85	− 4.9	85	142
	W	2.7	0.210	32	−21.1	45	161
3	D	4.0	0.024	92	− 0.9	92	154
	W	3.8	0.101	58	− 4.5	59	210
4	D	2.8	0.036	81	− 5.8	81	134
	W	2.0	0.262	26	−24.7(250)	42	150
5	D	3.5	0.039	89	− 2.9	89	148
	W	3.8	0.25(200)	18(200)	− 7.1(200)	29	104
6	D	3.0	0.25(100)	26(100)	0.0(100)	15	25
	W	2.3	0.46(50)	18(50)	− 0.3(50)	5	18
7	D	2.0	0.51(50)	20(50)	− 0.3(50)	5	8
	W	1.5	0.44(25)	18(25)	+ 1.3(25)	2	7
10	D	4.0	0.027	82	− 9.2	82	136
	W	3.6	0.15(50)	22(50)	− 1.1(50)	7	25
11	D	2.8	0.096	42	−12.5	48	80
	W	2.0	0.29(200)	33(150)	−22.3(200)	24	86
12	D	3.5	0.190	18	− 6.9	43	72
	W	1.5	0.59(50)	27(50)	+ 1.3(50)	7	25
8	D	1.5	0.082	60	−10.4	60	100
	W	1.7	0.20(200)	19(200)	−14.5(200)	28	100
			Air-entrained				
2	D	8.5	0.029	88	− 2.0	88	98
	W	6.5	0.033	90	− 3.4	90	94
3	D	7.0	0.019	94	+ 1.0	94	104
	W	8.0	0.021	89	− 0.5	89	93
4	D	7.0	0.044	73	− 7.6	73	81
	W	5.7	0.037	90	−15.4	90	94
5	D	7.7	0.026	94	− 0.3	94	104
	W	7.7	0.032	91	− 3.4	91	95
6	D	6.6	0.034	95	− 6.2	95	106
	W	6.0	0.040	95	− 3.5	95	99
7	D	7.0	0.190	61	− 5.9	61	68
	W	5.0	0.30(250)	34(250)	− 9.3(250)	41	43
10	D	8.0	0.018	90	− 9.1	90	100
	W	6.6	0.22C‡			45‡	47
11	D	6.7	0.19(250)	26(250)	− 9.0(250)	41	46
	W	4.8	0.100‡			60‡	63
12	D	6.6	0.032	89	− 5.1	89	99
	W	4.4	0.21(250)	27(250)	−13.9(250)	36	38
8	D	4.5	0.025	90	− 1.4	90	100
	W	5.7	0.024	96	− 1.1	96	100

*D = air dried. W = soaked 18-24 hr.
†Relative to Aggregate 8.
‡Extrapolated values.
Note: Numbers in parentheses are cycles at which tests were discontinued.

TABLE 6 — RESULTS OF FREEZING AND THAWING TESTS: 4500-PSI CONCRETES

Aggregate No.	Condition*	Air content, percent	Expansion at 300 cycles, percent	Percent of original E at 300 cycles	Change in weight at 300 cycles, percent	Durability factor at 300 cycles	Relative durability factor at 300 cycles†
			Non-air-entrained				
2	D	3.0	0.036	86	− 3.5	86	232
	W	2.5	0.16 (175)	21 (175)	− 1.2 (175)	23	96
3	D	2.6	0.022	90	+ 0.8	90	243
	W	2.2	0.43 (150)	23 (150)	+ 1.7 (150)	20	83
4	D	3.1	0.040	83	− 3.5	83	224
	W	1.5	0.68 (275)	59 (275)	−11.4 (275)	50	209
5	D	2.8	0.073	74	− 1.3	74	200
	W	2.1	0.2[illegible] (100)	22 (100)	+ 1.2 (100)	16	67
6	D	2.6	0.17 (175)	47 (175)	− 2.2 (175)	28	76
	W	2.7	0.07 (75)	76 (75)	+ 0.8 (75)	12	50
7	D	1.5	0.33 (75)	26 (75)	+ 1.3 (75)	7	16
	W	1.6	0.48 (75)	10 (75)	+ 1.1 (75)	7	29
10	D	1.7	0.17 (250)	29 (250)	− 1.3 (250)	41	111
	W	1.5	0.18 (50)	25 (50)	+ 3.0 (50)	8	33
10X‡	D	2.3	0.121	52	− 7.4	52	140
	W	1.6	0.16 (50)	30 (50)	+ 1.2 (50)	8	33
11	W	2.1	0.62 (150)	18 (150)	− 5.2 (150)	21	88
12	W	2.8	0.16 (50)	41 (50)	+ 0.5 (50)	8	33
8	D	1.0	0.23 (250)	13 (250)	− 5.5 (250)	37	100
	W	0.7	0.30 (150)	25 (150)	− 3.2 (150)	24	100
			Air-entrained				
2	D	7.5	0.021	91	− 2.2	91	111
	W	6.5	0.026	93	− 1.4	93	100
3	D	5.7	0.013	93	+ 0.9	93	114
	W	6.0	0.017	94	+ 1.1	94	101
4	D	6.5	0.330§	71	− 3.4§	71	87
	W	5.4	0.029	82	− 5.7	82	88
5	D	6.1	0.020	97	+ 0.9	97	118
	W	5.5	0.025	95	+ 0.2	95	102
6	D	6.5	0.015	98	− 1.8	98	120
	W	5.7	0.013	99	− 1.0	99	106
7	D	5.2	0.073	80	− 0.6	80	98
	W	4.9	0.133	64	− 1.7	64	69
10	D	5.2	0.012	93	− 1.0	93	114
	W	4.6	0.017	92	− 2.8	92	99
10X‡	D	5.3	0.018	91	− 2.3	91	111
	W	5.4	0.022	91	− 4.9	91	98
11	W	7.1	0.085§			48§	52
12	W	6.5	0.068	??	− 4.9	77	83
8	D	5.0	0.049	82	+ 0.6	82	100
	W	4.6	0.037	93	+ 0.3	93	100

*D = air dried. W = soaked 18-24 hr.
†Relative to Aggregate 8.
‡Repeat tests of Aggregate 10, using a lower cement content.
§Extrapolated values.
Note: Numbers in parentheses are cycles at which tests were discontinued.

the soaked aggregates, however, marked increases in durability were attained by the use of entrained air. For example, for the 3000-psi concretes shown in Fig. 1, non-air-entrained concrete made with Aggregate 6 in the soaked condition had a durability factor of 5, whereas with entrained air the durability factor was 95. Similarly, the respective durability factors for Aggregate 7 in the soaked condition were 2 and 41.

These data indicate that, for concretes of low durability, substantial increases in durability were effected by the use of intentionally en-

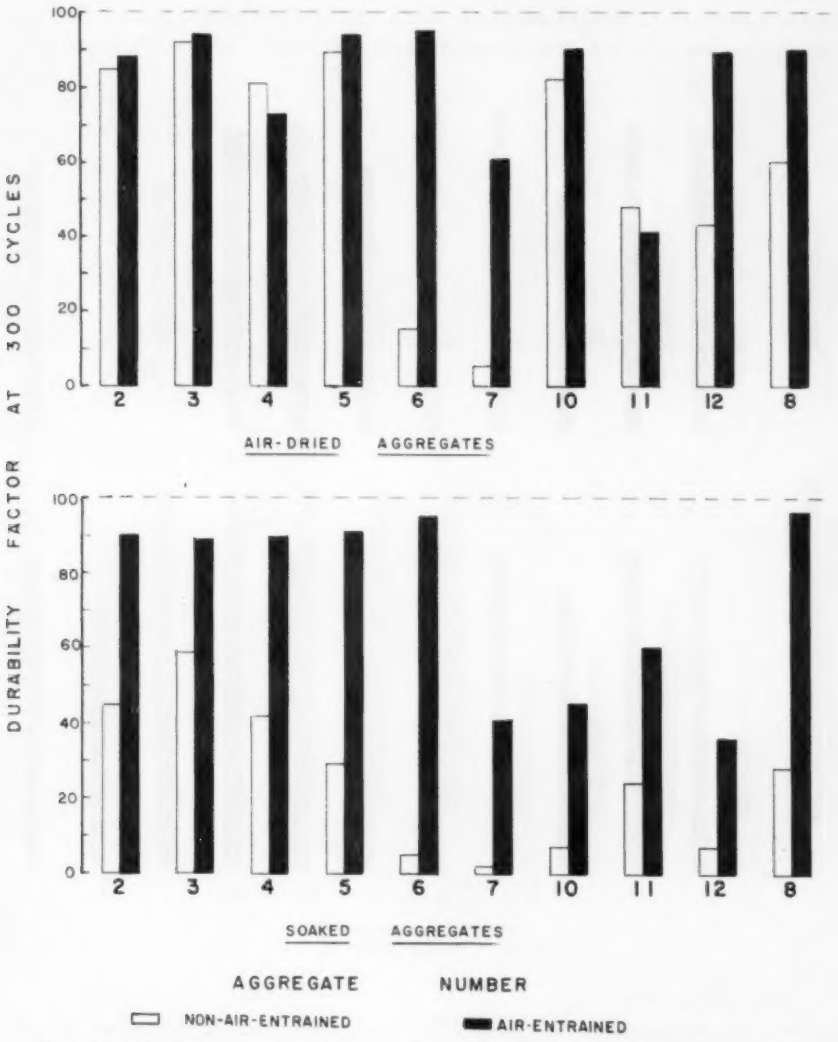

Fig. 1—Influence of air entrainment on durability of 3000-psi concretes

trained air in amounts similar to those used in concretes made with normal weight aggregates.

Influence of moisture content of aggregates on durability

Fig. 3 and 4 show these same individual durability factors rearranged to afford an easy comparison relative to the influence of moisture content of the aggregate on durability. Of the 19 possible non-air-entrained concrete comparisons included in these two figures, all of the concretes made with air-dried aggregates showed greater durability than the non-air-entrained concretes made with the soaked aggregates. For example, in the 3000-psi non-air-entrained concrete made with Aggregate

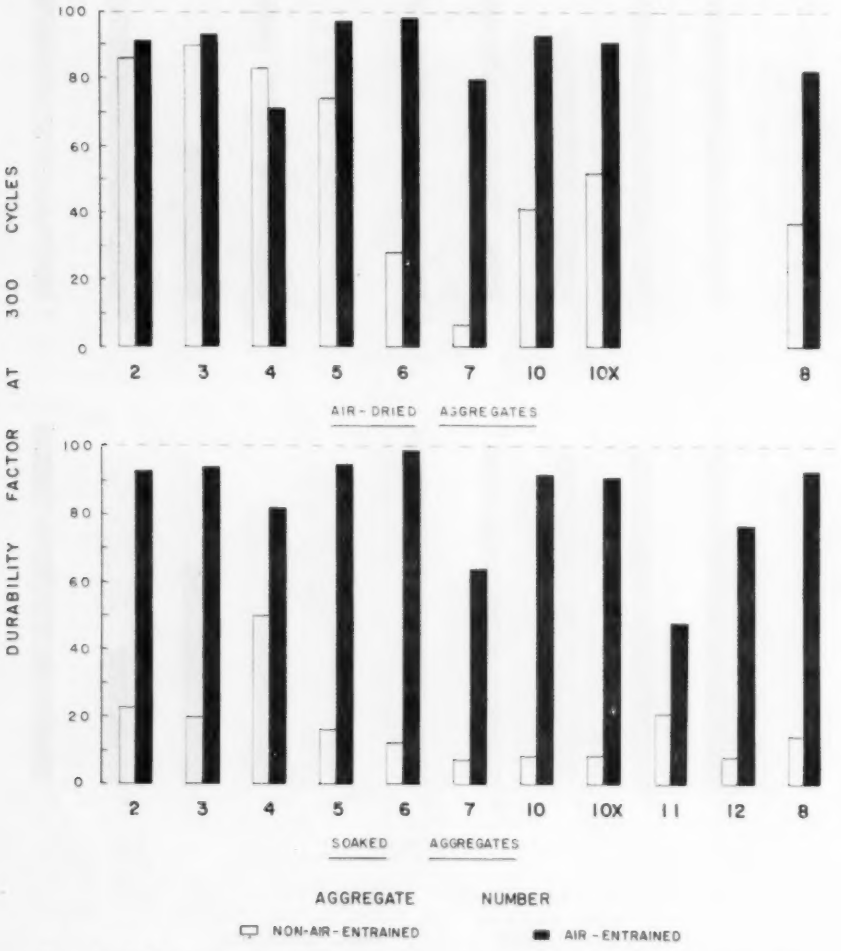

Fig. 2—Influence of air entrainment on durability of 4500-psi concretes

10, the durability factor for the soaked aggregate was 7 whereas the durability factor for the air-dried aggregate was 82, a very marked difference.

For the air-entrained concretes, concretes made with nine of the 19 aggregates showed slightly greater durability in the soaked condition than when air dried. In all but one of the nine cases, Aggregate 11 at 3000 psi, the durability factors were already at a high level and there-

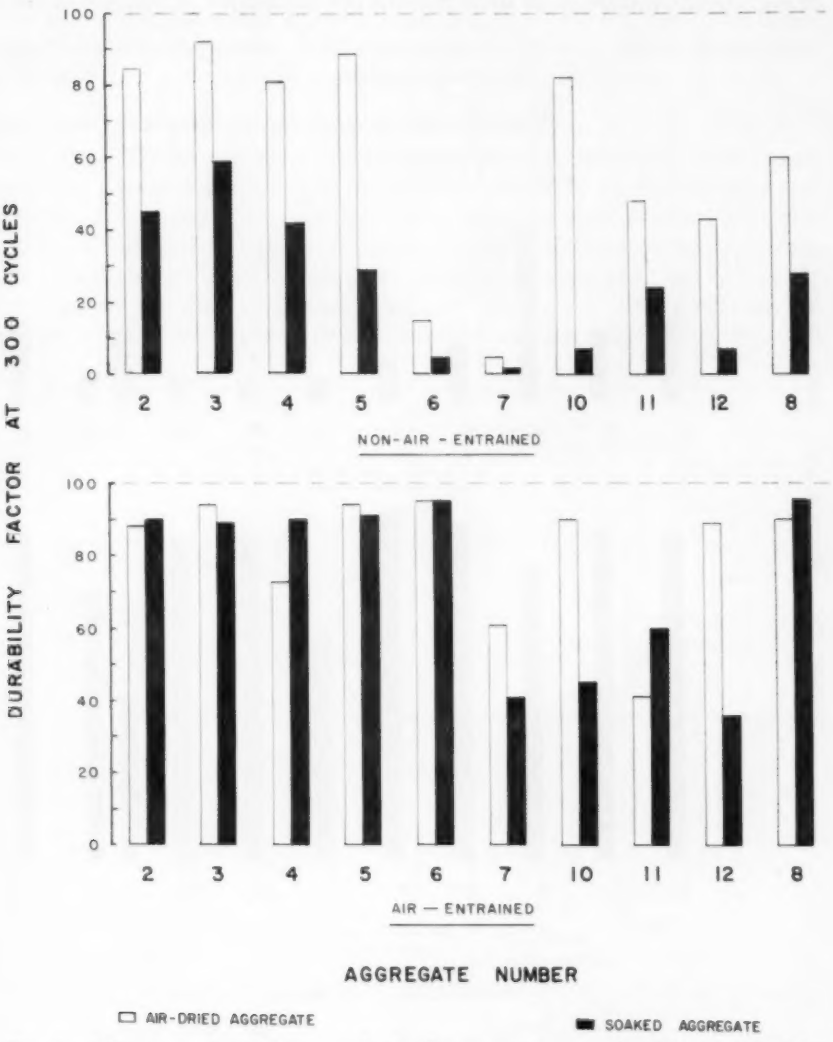

Fig. 3—Influence of moisture content of aggregates on durability of 3000-psi concretes

fore these small differences were probably not significant. In the case of Aggregate 11, the durability factor for the soaked aggregate was 60 as against 41 for the air-dried aggregate. This particular behavior does not appear to be explainable in terms of air content or any of the other measured properties of these concretes.

Influence of strength on durability

For the non-air-entrained concretes, strength did not appear to be of significant influence in determining the resistance of these concretes

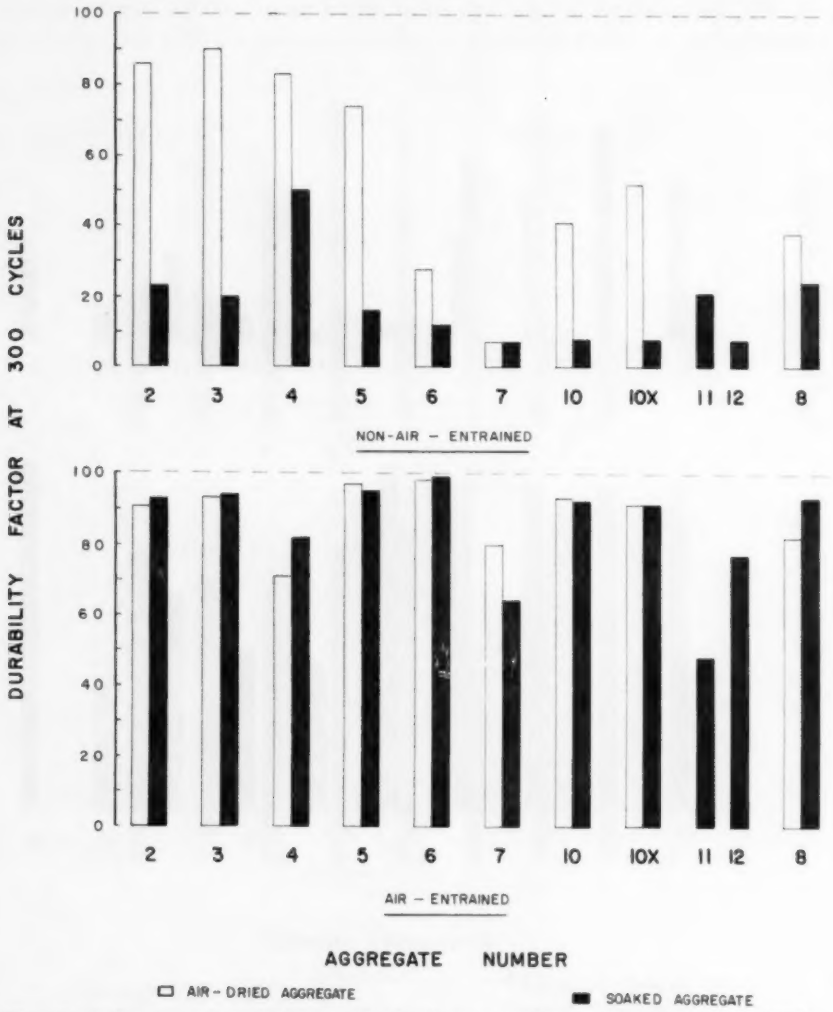

Fig. 4—Influence of moisture content of aggregates on durability of 4500-psi concretes

to freezing and thawing. Some aggregates showed significantly higher durabilities at 3000 psi concrete strength than at 4500 psi. On the other hand, the reverse was true for an equal number of aggregates, although the differences were not as great. The leaner mixes in these non-air-entrained concretes generally had somewhat higher air contents than the richer mixes and this may be the reason for some of the differences noted.

For the air-entrained concretes, the durability factors of the 4500-psi concrete were generally greater than for the 3000-psi concrete made with the same aggregate. This was particularly marked for Aggregates 7, 10, and 11 when used in a soaked condition.

Influence of absorption of concrete on durability

For the 4500-psi concretes, absorption was determined in accordance with the procedure set forth in ASTM C 330-53T, on lightweight aggregates for structural concrete. This tentative specification includes a table of maximum values of absorption for different strength levels. The reasons for these limitations were not apparent, but probably one was an attempt to insure durability through a limitation on absorption. For the highest strength listed in the table, 4000 psi, the

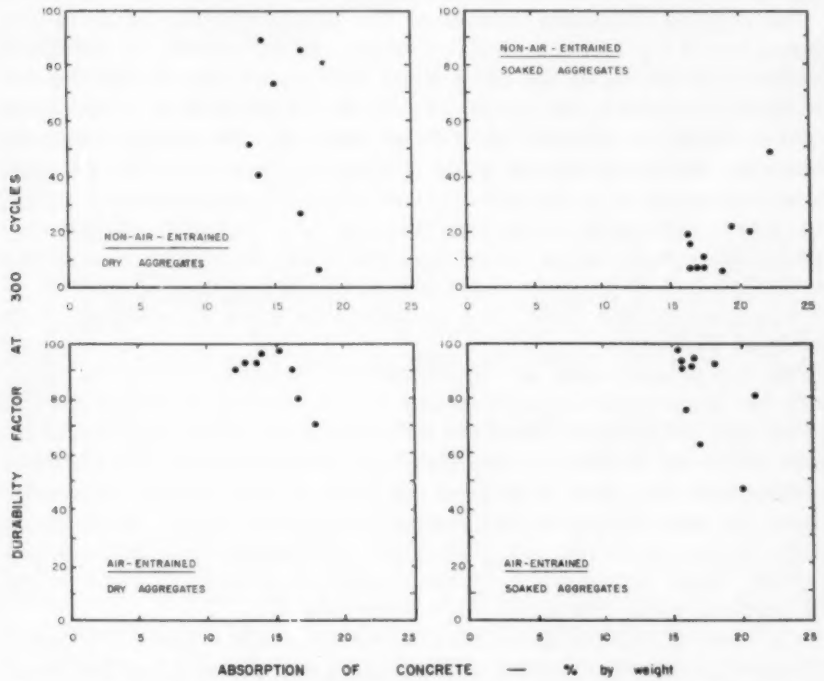

Fig. 5—Relationship between absorption of concrete and durability factor

absorption is limited to a maximum of 15 percent. Extrapolating to 4500 psi, the maximum absorption would be 13.5 percent. The concretes in this portion of this study ranged in strength from 3940 psi to 6510 psi.

Fig. 5 shows the relationships between the absorptions of the concretes made with the lightweight aggregates and their durability factor. It is apparent from these data than an absorption limitation is not a valid means of securing concrete resistant to freezing and thawing. The absorptions of the concretes containing entrained air are quite similar to those for the non-air-entrained concretes, yet the durabilities are markedly different.

Durability relative to normal weight aggregate

The normal weight aggregate used in these tests from Elgin, Ill., has a good service record in field performance. The sand is predominantly quartz. The coarse aggregate is well rounded, and is about half calcareous and half siliceous material. The aggregate contains chert which is troublesome in the large sizes, but in these tests the maximum size used was ¾ in. thereby moderating the influence of the chert on the durability of these laboratory specimens. All things considered, this aggregate would be classed as satisfactory.

The relative durability factors of the lightweight concretes ranged from a low of 7 percent to a high of 243 percent for the non-air-entrained concretes, as shown in the last column of Tables 5 and 6. For the air-entrained concretes, the range was 38 to 120 percent, a considerably smaller range in relative durabilities than for the non-air-entrained concretes. These results are quite similar to what would be obtained in a comparison of a number of normal weight aggregates of different types. Aggregate properties influence the durability of the concrete. The use of air entrainment raises the durability of concrete markedly, but these same aggregate properties may still operate to produce measurable differences in durability even at relatively high levels of durability.

This comparison does not mean that air-entrained concretes made with the lightweight aggregates which had relative durability factors lower than 100 percent would not be suitable for use in exposed structures subjected to freezing and thawing. It does indicate the probable relative standing with regard to this one normal weight aggregate. There are undoubtedly normal weight aggregates, which, although inferior to this particular one, still have satisfactory field performance records. These would similarly show relative durability factors below 100 percent, when compared with Aggregate 8.

It is most difficult to establish a minimum relative durability factor considering only one normal weight aggregate as the basis for these relative values. It appears more desirable and defensible to establish

TABLE 7 — RESISTANCE TO DEICER SCALING

Aggregate No.	Moisture condition of aggregate	Cement content, sack per cu yd	Slump, in.	Air content, percent	Scale rating at indicated number of cycles		
					100	200	300
3	D*	7.1	3.1	7.8	0+	1—	1—
	W	7.0	5.0	8.5	1+	1+	1+
4	D	5.8	2.9	7.1	1+	2+	3—
	W	5.7	2.8	7.9	2+	3+	4—
10	D	5.8	3.2	5.5	1—	1+	1+
	W	5.8	3.0	5.6	2+	3—	3+

*D = air dried. W = soaked 18-24 hr.

a minimum durability factor for the particular concrete, say a value of 70 based on 300 cycles of freezing and thawing. If 70 were the durability factor selected, two of the aggregates in 3000-psi air-entrained concrete would not meet this requirement in either the air-dried or soaked condition, and an additional two aggregates would not meet this requirement when used in a soaked condition. For the 4500-psi air-entrained concretes, two of the aggregates would fail to meet this requirement when used in a soaked condition, while all of the aggregates used in the air-dried condition would meet this requirement.

Resistance to deicer scaling

Air-entrained concretes made with three of the lightweight aggregates were used in laboratory tests to determine the influence of commercial flake calcium chloride as a deicer. These concretes were made at a later date than the original group and duplicated the 4500-psi air-entrained concretes for the aggregates in both the air-dried and soaked condition. The physical characteristics of these concretes and the surface scale ratings through 300 cycles of test are shown in Table 7. Concrete with the normal weight aggregate was not repeated for these tests, since a considerable background of laboratory data was available for this particular aggregate and other normal weight aggregates.

Non-air-entrained concretes made with normal weight aggregates generally reach a scale rating of 5 at about 50 to 75 cycles of test. Air-entrained concretes made with these same aggregates will show surface scale ratings between 1 and 2 at 175 cycles of test, and between 1 and 3 at 300 cycles of test. On this basis, Aggregate 3 would appear to be satisfactory in either the air-dried or soaked condition for an exposure involving the use of deicers. Similarly, Aggregates 4 and 10 when used in the air-dried condition would appear to be satisfactory, but possibly not when used in the soaked condition.

SUMMARY

Laboratory freezing and thawing tests were performed on non-air-entrained and air-entrained concretes made with nine different lightweight aggregates and one normal weight natural sand and gravel. Each aggregate was used in both an air-dried condition and a soaked condition, and in concretes at two different strength levels. In addition, three of the lightweight aggregates were used in air-entrained concretes which were tested for resistance to surface scaling resulting from the use of a deicer.

On the basis of these tests, the following observations seem appropriate:

1. The use of intentionally entrained air results in a marked increase in resistance to freezing and thawing of concretes made with lightweight aggregates.

2. The amount of intentionally entrained air required for adequate durability of concrete made with lightweight aggregates is similar to that required for normal weight aggregates.

3. The moisture condition of the aggregate has a significant influence on the resistance to freezing and thawing of the concrete. This is particularly so for the non-air-entrained concretes and for certain aggregates in the leaner air-entrained concretes.

4. For three of the four lightweight aggregates which, in a soaked condition, produced 3000-psi concretes of relatively low durability, significant increases in durability resulted when used in lower water-cement ratio, richer concretes.

5. The spread in durability among the concretes made with the different lightweight aggregates appears no greater than might be encountered with normal weight aggregates.

6. Concretes made with lightweight aggregates can be made as resistant to the effect of deicers by the use of entrained air as concretes made with normal weight aggregates.

7. Aggregate properties are obviously of importance in determining the level of durability, even in air-entrained concretes. In lieu of precise knowledge as to the degree of influence of factors such as porosity, pore structure, and others, it appears desirable to evaluate these effects by means of laboratory freezing and thawing tests, supplemented by field performance records. This is analogous to the procedure presently followed for normal weight aggregates.

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American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

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11-Year Study of Concrete Stave Silo Durability

In 1940 ACI Committee 714, then titled "Recommended Practice for the Construction of Concrete Silos" C. A. Hughes, chairman, established a long range field experiment in concrete stave silos. This research was to provide guidance for preparing recommended standards for construction of farm silos. Data on the test silos were collected and tabulated largely by personnel of the Portland Cement Association and the National Silo Association. The result of the committee work was this report which summarizes 11 years of field tests and additional laboratory tests made on the staves after the silos were dismantled in 1955. Valuable assistance in tabulating and analyzing the data was given by M. L. Burgener, W. D. Hanford, and J. J. Shideler of the Portland Cement Association.

Serving on the committee to prepare this report were:

Thomas E. Long, Portland Cement Association, New York, N.Y., Chairman

W. W. Gurney, Madison Silo Co., Madison, Wis.

G. A. Spry, Waterloo Concrete Co., Waterloo, Iowa

C. F. Rogers, Agricultural Consultant, Wooster, Ohio

C. C. Carlson, Portland Cement Association, Chicago, Ill.

T. A. Meyer, National Silo Association, Louisville, Ky.

In 1940, ACI Committee 714 set up a research program to test the adequacy of a proposed ACI standard, "Recommended Practice for the Construction of Concrete Farm Silos." The test consisted of four silos erected on the Ohio Agricultural Experiment Station at Wooster, Ohio. Each of the silos contained dry-tamped and wet-cast staves made from concrete mixes containing three grades of aggregate, three levels of cement content, and two types of cement. In addition to these variables, the interiors of two of the silos were coated with a portland cement "wash-coat" while the other two were left uncoated. The silos were used for storing silage under conditions similar to those found in average farm service.

Periodic visual observations of the conditions of the staves were made during the 11-year test period. Flexural strength and absorption tests of the staves, performed on unexposed staves near the start of the project and also on 11-year old staves removed from the silos, showed good correlation with the visual results. The study showed that to obtain the desired low absorption and high strength values, silo staves must be made from concrete containing good aggregate and moderately high cement content. Proper application of a portland cement wash coat will substantially increase the service life of a quality concrete stave silo.

The tests specified for concrete staves in ACI 714-46 are good yardsticks for measuring the durability of concrete silos in service; however, results of this study indicate that a more restrictive current standard would result in prolonged life of the staves under service conditions.

Fig. 1—Four concrete stave silos used to test durability of staves under actual service conditions. Erected in 1944 on the Ohio Agricultural Experiment Station, Wooster, Ohio

■ CONCRETE SILOS ASSEMBLED FROM PRECAST STAVES have been used for the storage of forage crops on American farms for many years. These silos may range in diameter to 30 ft and in height to 60 ft or more. Many of the silos on the farm today are used for storage of grains as well as for silage.

Farm service conditions may subject the staves alternately to freezing and thawing, wetting and drying, varying bending stress, and attack by corrosive fluids. As a basis for obtaining staves that would endure under these varied conditions, the American Concrete Institute, through Committee 714, established a minimum limit for average flexural strength of 690 psi and a maximum average for absorption of 6 percent. These specifications, known today as ACI 714-46, have been generally accepted, but there has always been some question of their adequacy as a yardstick for determining concrete stave durability.

In 1940, ACI Committee 714 established a concrete stave silo durability project at the Ohio Agricultural Experiment Station, Wooster, Ohio. The purpose of this research project was to determine the relationship between short term laboratory tests on concrete staves and the actual service performance of concrete stave silos.

The major feature of the project consisted of four concrete silos (Fig. 1) erected in May, 1944. These silos were filled with silage each year until 1955 when it became necessary to dismantle them although their condition was still good. Each silo contained 24 columns of 13 staves. Each of the 24 columns represented one of the 24 variables involved in the project. Aggregate quality and cement content were the principal variables in the concrete. Half of the staves were manufactured by the dry-tamped method and half by the wet-cast method.

VARIABLES IN PROJECT

Aggregate quality

Three qualities of aggregate were used in the project (Table 1). The quality of the aggregate was determined by the weight loss in the sodium sulfate soundness test described in the American Society for Testing Materials tentative method C 88-39T. The aggregate rating "good" exhibited a weight loss of 1.6 percent in this test. The "poor" aggregate showed a weight loss of 19.5 percent. Also, this aggregate contained considerable amounts of chert, shale, and other materials believed to be detrimental to concrete durability. To obtain the "mixed" or intermediate quality aggregate, equal proportions, by weight, of good and poor aggregate were mixed together.

Yield per bag of cement

The number of 2½ x 10 x 30-in. staves produced per bag of cement was 7, 10, and 13 (Table 1). The wet-cast staves, being larger in size—2½ x 12¼ x 30 in.—actually yielded only 5.6, 8.0, and 10.4 staves per bag of cement. However, for the sake of simplifying identification, the yield figures are stated throughout the report in terms of 2½ x 10 x 30-in. equivalent staves. The 7, 10, and 13 staves per bag mixes required cement contents of 8.9, 6.2, and 4.8 bags per cu yd of concrete, respectively.

TABLE 1 — CONCRETE MIXES FOR DURABILITY SILOS

Quality of aggregate	Yield* per bag of portland cement	Type of portland cement
Good Mixed Poor	7 Staves (8.9 bags per cu yd)	I
Good Mixed Poor	10 staves (6.2 bags per cu yd)	I
Good Mixed Poor	13 Staves (4.8 bags per cu yd)	I
Good Mixed Poor	13 Staves (4.8 bags per cu yd)	Ia

*Based on staves measuring 2½ x 10 x 30 in. (0.43 cu ft concrete volume).

TABLE 2 — CLASSIFICATION FOR VISUAL RATING OF STAVES

Class	Description
O	Surface apparently unaffected
I	Staves lightly etched
II	Etching progressed 1/16 in. to ⅛ in. deep
III	Etching progressed ⅛ in. to ¼ in. deep
IV	Extensive and deep etching to ½ in. deep

Types of cement

Two types of portland cement were used: Type I and Type Ia (Table 1). Type I cement complied with ASTM C 150-41. Type Ia was of the same composition except that an air-entraining agent (0.04 percent vinsol resin) was interground during manufacture.

Molding procedures

Two molding procedures were used: dry-tamped and wet-cast methods. The dry-tamped staves were molded on a tamping machine from semidry nonplastic mixes, while the wet-cast staves were molded on a vibrating table from plastic mixes of a rather stiff consistency.

Protective coating (Application and curing)

In addition to the other variables, a portland cement "wash coat" was applied to the interior of two of the silos (Silos II and IV) as a protective coating against silage attack. The wash coat consisted of portland cement and water mixed to a heavy cream consistency. This mixture was applied with a stiff bristled brush.

Simulating possible field conditions, one silo (No. II) was coated during a hot, dry period. The interior surface of the staves was slightly dampened prior to the application of the wash coat, and all procedures

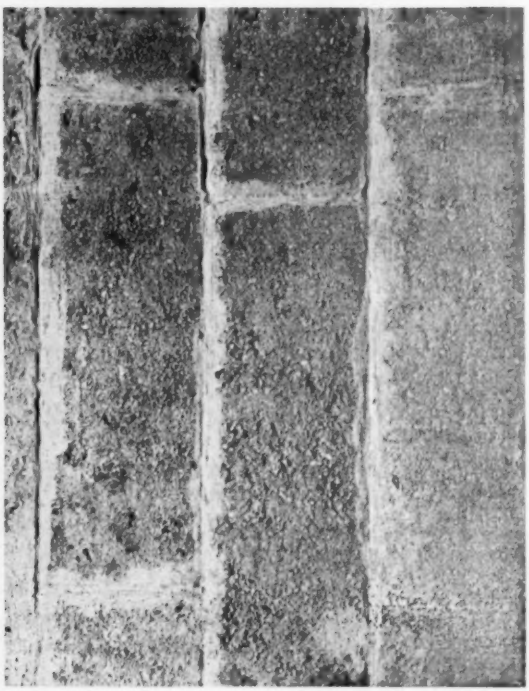

Fig. 2—Partial view of interior surface of a silo showing condition of uncoated 10 x 30-in. dry-tamped staves after 6 years of exposure to silage

for curing were omitted. Silo IV was treated during a period of damp, rainy weather which provided ample pre-wetting of the receiving surface and excellent atmospheric conditions for curing the wash coat.

TEST PROCEDURES

Visual observations

After 3, 5, 6, and 11 years of service, the surface conditions of the bottom nine staves in each column of each silo were visually rated by a team of observers. Five classifications of surface condition were established and are shown in Table 2.

Photographs showing the surface condition of the staves were taken after 6 years of exposure to silage. Staves in courses No. 3, 4, and 5 in each column were photographed. These courses were located slightly below and above ground level, which was the area showing the greatest deterioration. The typical appearance of 6-year old uncoated staves is shown in Fig. 2 and 3.

Laboratory tests

In 1946, laboratory tests were conducted on 120 uncoated staves from the 24 variables. These staves were companions to those used in the silos. They were tested after a period of approximately 2 years storage

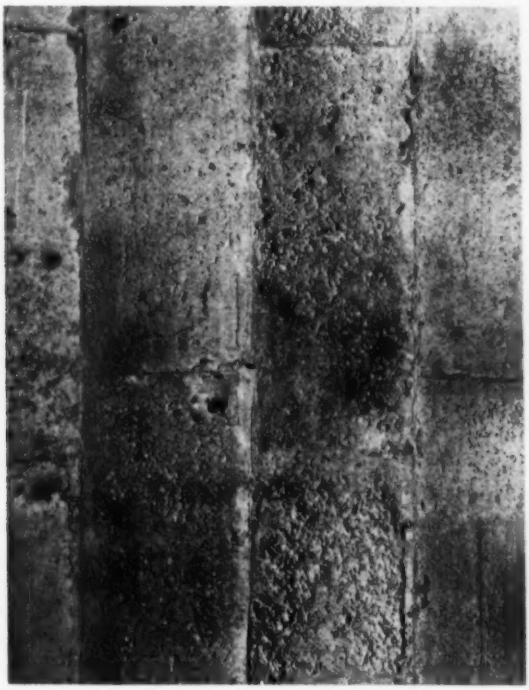

Fig. 3—Partial view of interior of a silo showing surface condition of uncoated 12¼ x 30-in wet-cast concrete staves after 6 years of service

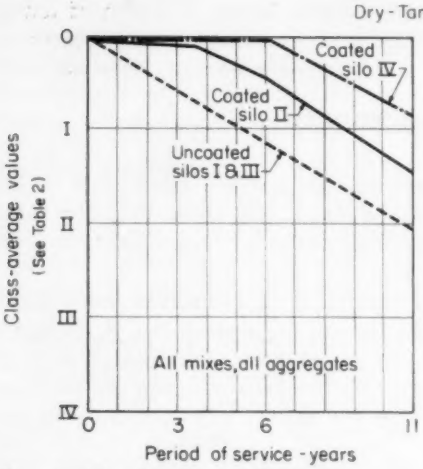

Fig. 4—Visual ratings of interior surface condition of coated and uncoated dry-tamped staves during 11 years of exposure to silage

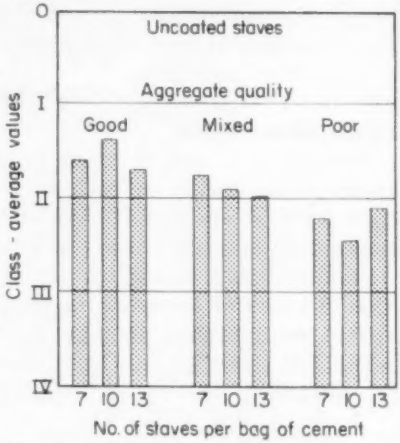

Fig. 5—Visual rating of interior surface of dry-tamped staves showing effect of cement content and aggregate quality after 11 years service

in a cool dry place. The staves served as standards of comparison for later laboratory tests.

Additional staves from each group were stored outdoors for the entire 11-year test period. These were subjected to weather but not to silage attack.

When the silos were dismantled, staves taken from courses 3, 4, and 5 of each of the silos were tested. This permitted a comparison between unexposed staves, those exposed to the weather, and those staves subject to silage attack.

Tests for flexural strength and absorption were performed as specified in ACI 714-46. Absorption values for the exposed staves were computed on the basis of 2½ in. nominal thickness.

Soniscope tests were made to determine the pulse velocity through the staves. The soniscope was used in accordance with procedures described in Portland Cement Association *Bulletin* No. 36, "Soniscope Tests Concrete Structures." Tests for the resonant flexural and torsional frequencies and the calculations for the modulus of elasticity in both flexure (E) and torsion (G) were obtained according to procedures described in ASTM C 215-55T.

The dynamic tests described above were initiated to evaluate new nondestructive testing techniques in terms of established testing procedures. If these procedures were found to be satisfactory, the pulse velocity method in particular might be used to quickly "spot" test the quality of concrete staves before they were erected in a silo. These tests were conducted on exposed staves only.

PROJECT RESULTS WITH TYPE I CEMENT

Dry-tamped staves

Visual — The results of the visual ratings are shown in Fig. 4 and 5. Fig. 4 compares stave condition with years of service. The graph indicates that the rate of surface attack is almost constant, but attack on the staves is delayed by the wash coat. The uncoated staves had the lowest rating while the staves coated during the rainy period (Silo IV) had the highest rating. Staves coated during the dry period (Silo II) had an intermediate rating. This indicates that the better cured protective coating, as provided in Silo IV, would result in 6 years more service than an uncoated silo.

Fig. 5 compares stave condition with cement content and aggregate quality. Visual ratings of the uncoated exposed staves after 11 years of service, indicates that the staves made with good aggregates gave the best performance. Visual rating of dry-tamped staves was difficult due to the initial rough surface condition caused by the tamping procedure.

Laboratory tests: Flexural strength — Flexural test results on dry-tamped, uncoated staves from Silos I and III are shown in Fig. 6. All the staves, with one exception, tested higher in strength after 11 years

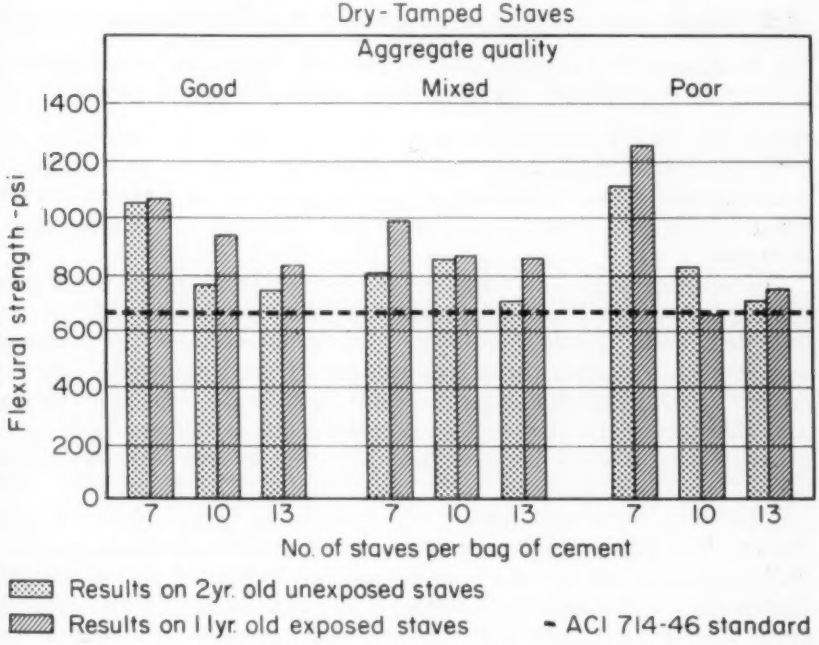

Fig. 6—Effect of cement content and aggregate quality on flexural strength of uncoated dry-tamped staves

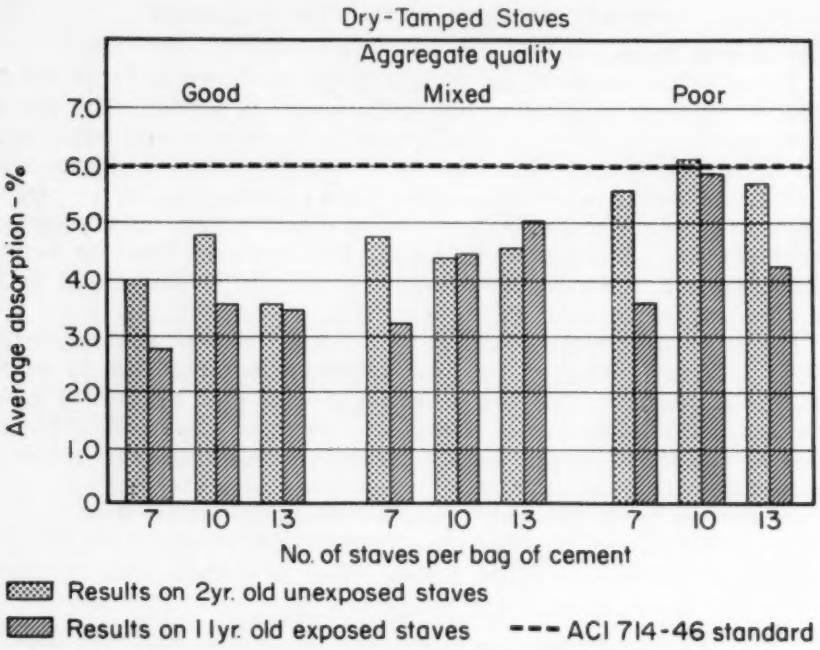

Fig. 7—Results of absorption tests of uncoated dry-tamped staves containing three grades of aggregate versus yield per bag of cement

use in the silo than unexposed 2 year old staves from the same mix. Only the staves made from the 10 staves per bag mix and containing poor aggregate had fallen below the ACI minimum flexural requirement of 690 psi. Staves made from the rich mixes had the highest strength.

The average flexural strength of the coated dry-tamped staves taken from Silos II and IV averaged approximately 20 percent higher than uncoated staves. While there was a wide range between the maximum and minimum values, the staves containing good aggregate and high cement content had the largest increase in strength. The average flexural strength of the 11 year old companion staves exposed only to the weather was about 5 percent less than that of the uncoated staves taken from the silos.

Laboratory tests: Absorption — A comparison of the absorption test results obtained on the dry tamped, uncoated staves is shown in Fig. 7. With one exception all of the two year old unexposed staves met ACI absorption requirement of less than 6 percent. In nearly all cases, the absorption of the 11 year old staves varied inversely with cement content and aggregate quality and was lower than the value for the 2 year old staves.

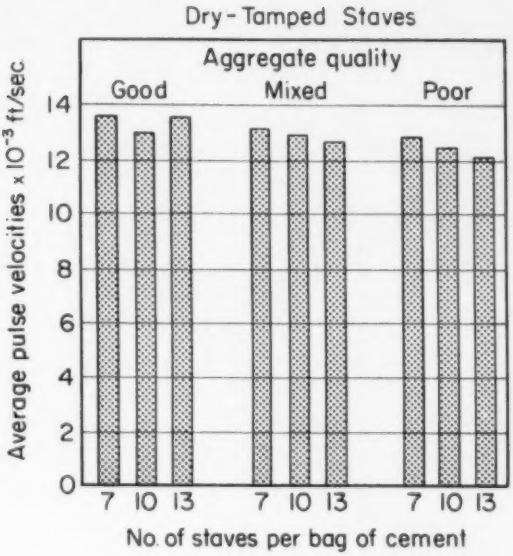

Fig. 8 — Soniscope test results versus yield per bag of cement for uncoated dry-tamped staves following an 11-year period of silage exposure

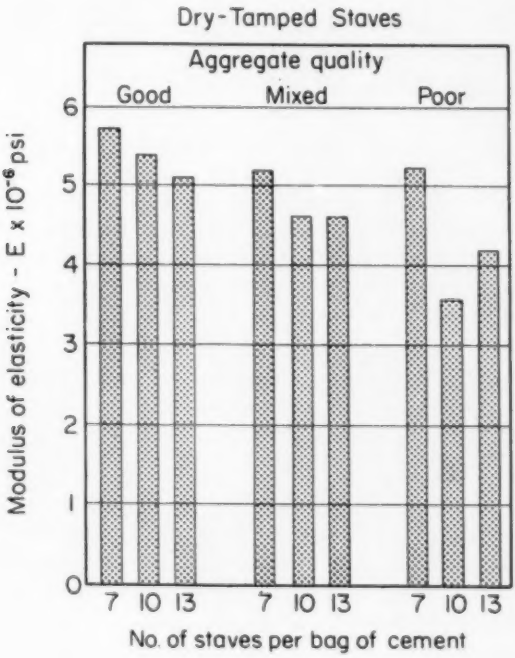

Fig. 9 — Effect of cement content and aggregate quality on modulus of elasticity test results for uncoated dry-tamped staves

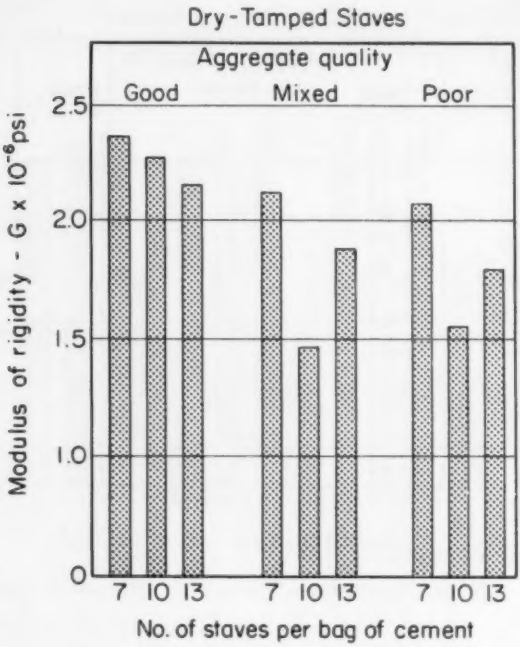

Fig. 10—Modulus of rigidity test results versus yield per bag of cement for uncoated dry-tamped staves after 11 years of service

Laboratory tests: Pulse velocity, modulus of elasticity, and modulus of rigidity — The results of these tests appear in Fig. 8, 9, and 10, respectively. Since these tests were not performed on the 2 year old unexposed staves, no direct comparative data are available. However, the results obtained do, in general, indicate the quality and durability of the concrete staves.

The modulus of rigidity and modulus of elasticity of the coated, dry-tamped staves after 11 years of exposure to silage averaged 15 to 20 percent higher than the uncoated staves. The staves from the silo having the better cured coating had a modulus of rigidity and modulus of elasticity averaging 25 to 30 percent higher than the uncoated staves. The companion staves exposed to the weather, only, and tested at the same time as those from the silos, had a modulus of rigidity and modulus of elasticity averaging slightly higher than the uncoated staves exposed to silage.

Wet-cast staves

Visual — The results of visual ratings for wet-cast staves are shown in Fig. 11 and 12. The rate of surface decay, shown in Fig. 11 for the uncoated staves, was almost constant during the 11 years of exposure. The staves coated during the dry period (Silo II) showed no indication of pitting during the first 3 years of exposure. Then the stave surface appeared to deteriorate rapidly and at the end of 6 years of service

had the same rating as the uncoated staves. A possible explanation for this occurrence is that deterioration of the stave surface may have been taking place beneath the coating.

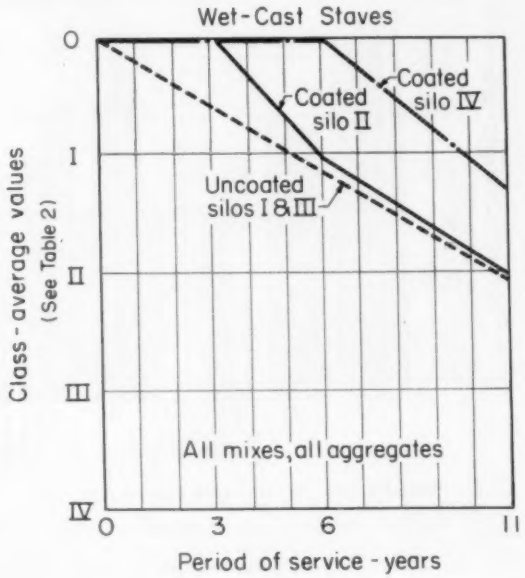

Fig. 11 — Visual ratings of interior surface condition of coated and uncoated wet-cast silo staves during 11 years of exposure to silage

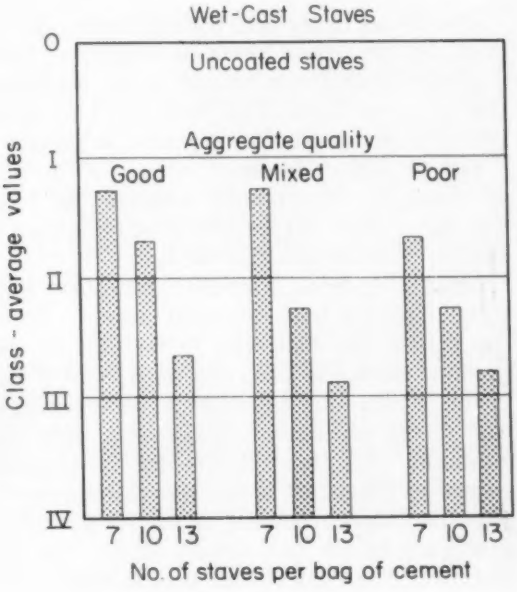

Fig. 12—Visual rating of interior surface of wet-cast concrete silo staves showing effect of aggregate quality and cement content after 11 years service

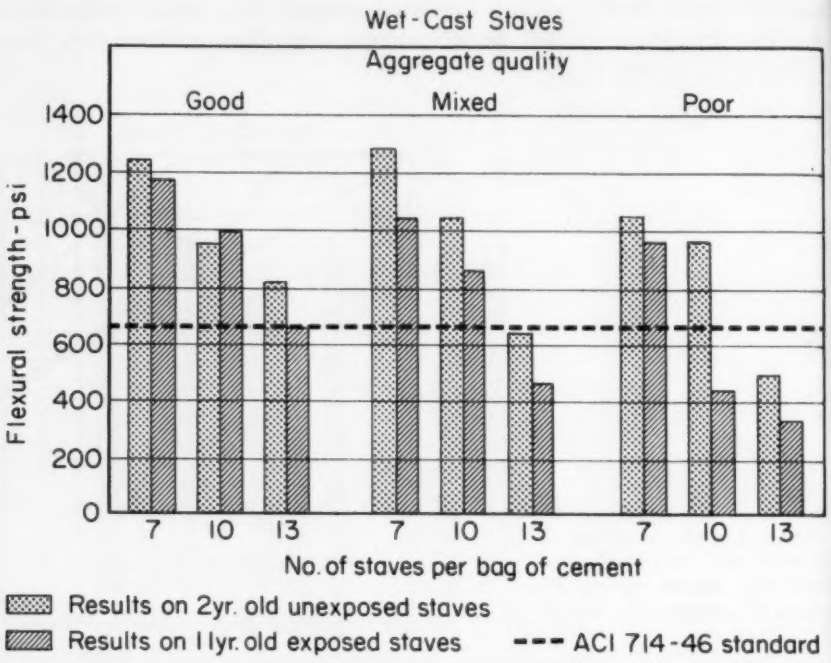

Fig. 13—Effect of aggregate quality and cement content on flexural strength of unexposed wet-cast-concrete staves

The staves in the silo coated during the rainy period (Silo IV) rated high for the first 6 years of exposure. For the balance of the 11-year exposure period, the rate of decay was approximately the same as for the uncoated silos. This indicates that a moist-cured wash coat would result in an additional 6 years of service over an uncoated silo.

Fig. 12 shows the visual ratings of the uncoated staves after 11 years of exposure. The results indicate the importance of a rich mix for wet-cast staves. Aggregate quality is also an important factor in obtaining durable wet-cast staves.

Laboratory tests: Flexural strength — Fig. 13 shows flexural test results obtained with wet-cast staves. With one exception all of the test staves that had been in the silo for the 11-year period had lower flexural strengths than the unexposed 2 year old staves.

The 2 year old, unexposed, wet-cast staves from the 13 staves per bag mixes containing mixed and poor aggregate did not meet the minimum ACI flexural strength requirements. After 11 years of exposure to silage, all of the wet-cast staves of the 13 staves per bag mix plus the staves from the 10 staves per bag mix containing poor aggregate were below ACI specifications. Staves containing good aggregate and made from the richest mix showed a minimum decline of strength.

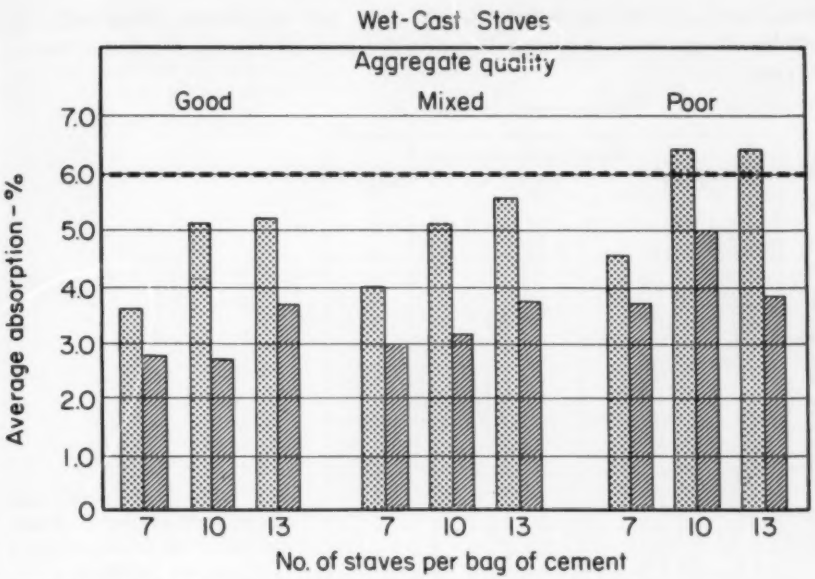

Fig. 14—Yield per bag of cement versus absorption for uncoated wet-cast concrete silo staves

The flexural strength of the coated wet-cast staves, from Silos II and IV, averaged approximately 25 percent stronger than the uncoated staves. However, the strength gained due to the wash coat varied over an extremely wide range — from a 10 percent decrease to approximately a 90 percent increase. The greatest difference occurred in the staves containing either a low cement content or poor aggregate. Although the coating benefited the poorer staves most, their flexural strength was still below or near the minimum limits of ACI-714. The average flexural strength of the 11-year old companion staves, exposed to weather only, was almost 30 percent greater than the uncoated staves exposed to silage.

Laboratory tests: Absorption — Fig. 14 shows a comparison of the absorption test results obtained with wet-cast, uncoated staves. Of the 2 year old staves, those containing poor aggregate and cast from 10 and 13 staves per bag mixes failed to meet ACI absorption requirements. All of the staves, after 11 years of exposure, had lower absorption values than similar staves tested after 2 years of storage.

Laboratory tests: Pulse velocity, modulus of elasticity and modulus of rigidity — The results of these tests appear in Fig. 15, 16, and 17, respectively. Since these tests were not performed on the 2 year old,

unexposed staves, no comparative data are available. However, the results, in general, indicate the quality and durability of the concrete staves.

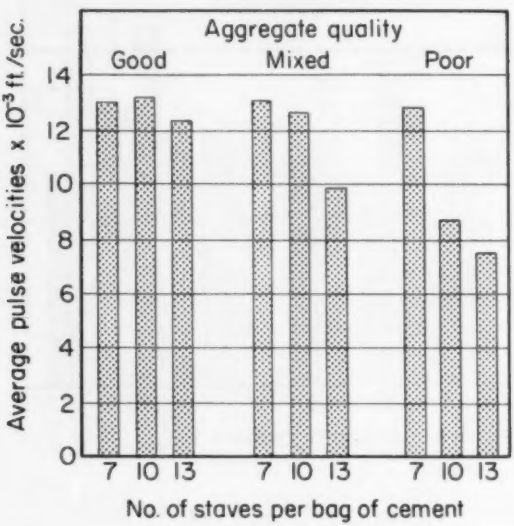

Fig. 15 — Results of soniscope tests of uncoated wet-cast staves following 11 years exposure

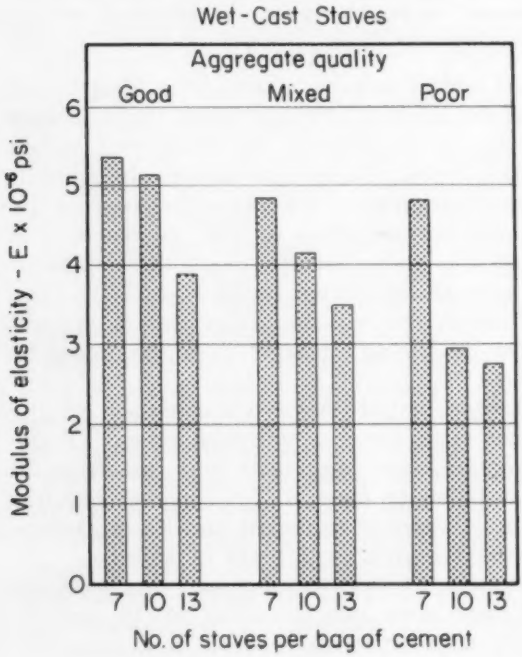

Fig. 16—Results of modulus of elasticity tests on uncoated wet-cast concrete silo staves after 11 years of exposure to silage

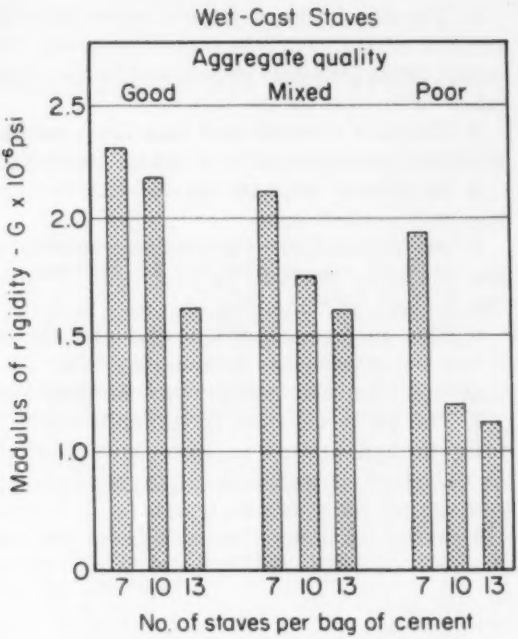

Fig. 17—Modulus of rigidity test results of uncoated wet-cast concrete silo staves after 11 years of exposure to silage attack

The modulus of rigidity and modulus of elasticity of the coated wet-cast staves after 11 years of exposure to silage averaged 10 to 15 percent higher than the uncoated staves. Staves from the silo having the better cured coating had a modulus of rigidity and a modulus of elasticity averaging 15 to 20 percent higher than the uncoated staves. The companion staves exposed to weather, only, had a modulus of rigidity and modulus of elasticity averaging about 20 percent higher than the uncoated staves exposed to silage.

PROJECT RESULTS WITH TYPE 1A CEMENT

Due to the variable results obtained and the limited scope of the tests (air-entraining cement was used in the lean mixes only) conclusions regarding the relative durability of air-entrained versus non-air-entrained concrete for dry-tamped and wet-cast silo staves could not be drawn.

CONCLUSIONS

On the basis of the preceding results, it is possible to draw the following conclusions:

1. The sodium sulfate test for soundness of aggregates (ASTM C 88-39T) appears to be useful in selecting sand and gravel aggregates for concrete silo staves.

2. The durability of the silo staves was directly related to the cement content of the concrete mixes involved. Staves from mixes producing seven units per bag of cement were superior in durability to those made from the leaner mixes.

3. Flexural strength test data from new and also from exposed staves correlate closely with the visual performance of staves in service.

4. In general the new staves with the lowest absorption values gave the best performance.

5. An application of a portland cement wash coat to the interior of the silo after erection is beneficial. Proper application and curing of the coating will increase the service life of a silo substantially.

6. The surface condition and the flexural strength of the staves when the study was terminated after 11 years of service, indicated that the silos had many years of useful service remaining.

7. The pulse velocity, modulus of elasticity, and modulus of rigidity tests, though limited in scope, did parallel the visual ratings as well as the flexural and absorption test results. These tests have the advantage of being nondestructive.

8. Based on data obtained from this research an increase in the flexural strength specification from 690 to 750 psi and a reduction in the absorption limit from 6 to 5 percent would have disqualified, prior to use, nearly all of the staves whose 11-year test results fell below current ACI requirements.

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Title No. 57-40

Gravel Beneficiation in Michigan

By FRANK E. LEGG, JR., and WILLIAM W. McLAUGHLIN

Michigan natural gravel is now competing with imported high quality coarse aggregates intended for concrete subject to severe winter exposure, as a result of installation of various types of beneficiation plants. Thus, local deposits are being utilized, after upgrading, without sacrifice of concrete quality. This development was prompted in large measure by the decision of the Michigan State Highway Department that aggregate quality standards would not be relaxed despite the greatly increased demand of the accelerated highway construction program.

Many of Michigan's lower peninsula glacial gravels have been found particularly amenable to upgrading by heavy media separation, although commercial installations of elastic fractionation, jigs, and soft particle disintegrators are being tried. At present, 14 beneficiation plants are operating in the State. Laboratory concrete freeze-thaw evaluations of the effectiveness of several of the plants are presented together with observations on routine field inspection. Caution is advised against undue optimism of one beneficiation process as opposed to another for all deposits — tailoring of the particular process to the needs of each aggregate source, together with consideration of economics involved, seems the wiser course of action.

■ WEATHERING ACTION ON HIGHWAY CONCRETE in the southern peninsula of Michigan is severe. Numerous examples have existed where concrete has been defaced by freezing and thawing in the presence of moisture from rain or snow, and by action of chloride salts used for ice removal. Exclusive use of air-entrained concrete for all exposed concrete for the past 15 years by the Highway Department has greatly reduced the scaling problem. In fact, appreciable scaling of air-entrained concrete, containing the required amount of air, and resulting from salt or freeze-thaw attack on the mortar or paste fraction, has not been experienced although this has occasionally been alleged. Examination of the hardened concrete in such cases has invariably shown a poor or nonexistent air void system. Despite careful control, occasional loads of non-air-entrained concrete have entered the work.

It should not be construed from these facts that all concrete problems in the area have been solved — particularly those associated with aggregates. Otherwise excellent concrete has been marred and the surface disfigured by deleterious particles popping out of the surface. Studies at Purdue University indicate certain coarse aggregates con-

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tribute to more deep-seated distress of the concrete, namely pavement blow-ups.[1] Michigan has likewise found over-all field performance of concrete to be related to laboratory freeze-thaw durability of the particular aggregate.[2] Aggravated cases of distress have been more prevalent in concrete not subject to Highway Department control. Sidewalks and driveways in housing developments are a frequent source of complaint by ordinary consumers. Maintenance of supplies of good coarse aggregate for exposed concrete has been a problem under study for many years, and solutions are being constantly sought by an alert and aggressive aggregate industry.

The lower peninsula of Michigan is unusual in being provided with the surrounding cheap water transportation of the Great Lakes and, as a consequence, importation of satisfactory aggregates, usually crushed limestone, is economically possible. However, users still are naturally inclined to wish to utilize local natural glacial gravel deposits which are abundant throughout most of the region.

The character of the gravel in these glacial deposits is extremely variable, sometimes even changing considerably within a single deposit. Petrographers examining these gravels invariably remark on the complexity of the rock types they are called upon to identify — hard igneous rocks, cherts, limestones, schists, soft shales, ochres, and iron claystones may be mixed into a single deposit. The Michigan State Highway Department Standard Specifications have long recognized that some of these rock types are deleterious in exposed concrete. The 1924 Michigan specifications required aggregates be "free of soft material or rock which readily disintegrate."[3] The 1926 edition itemized in greater detail the rock types considered deleterious and from the work of Emmons,[4] the 1934 edition first adopted a maximum permissible chert content for concrete coarse aggregate. Subsequent editions of the standard specifications have spelled out in ever greater detail the rock types considered deleterious, the most recent edition being that of 1957. The present specifications recognize three types of deleterious gravel particles:[5] (1) "soft particles include shale, soft standstone, ochre, coal, iron bearing clay, weathered schist, shells, floaters, partially disinte-

Fig. 1—Location of 14 beneficiation plants operating in Michigan during 1959

grated particles, and any other particles which are structurally weak or fail to meet the soundness test;" (2) "chert;" and (3) "hard absorbent particles." The latter term has grown to be something of a misnomer since it includes a variety of deleterious particles not specifically itemized under (1) and (2). Aggregate inspectors in the field and gravel producers alike, have grown to recognize the various types and borderline classification troubles are not too frequent. For normal exposed concrete, the "soft particles" are limited to 3 percent and the sum of the three types to either 8, 9, or 10 percent depending upon the maximum size of the aggregate. For the highest quality exposed architectural concrete, the soft particles may not exceed 1 percent and the sum of all three types may not exceed 3 percent.

GRAVEL BENEFICIATION PROCEDURES

The recently expanded highway construction program in Michigan has required a large increase in aggregate production with consequent temptation to lower quality standards to permit greater production from present sources or to allow using new deposits of marginal quality. A firm policy decision was made by the Michigan State Highway Department that expediency would not force lowering of standards. Thus, gravel producers, due to consumer demand, have been upgrading the quality of their product from present deposits and have been developing new pits formerly judged to be too low in quality to be considered. In so doing, they have been recently trying a number of special processes to provide a material which would consistently meet the highway department requirements. Some other user agencies in the state are likewise now demanding the premium beneficiated product.

Four types of beneficiation processes are being used: jig, elastic fractionation, cage mill disintegrator, and heavy media separation. Detailed description of three of these processes is given elsewhere[5]

and will not be repeated here. Success with these processes has ranged from poor to excellent. Fig. 1 shows the location of the plants which operated during the 1959 season.

Jigging

Jigging provides a gravity separation of gravel by "hindered settling" in water which is subjected to a continuous series of upward pulsations. The low gravity material will tend toward the top of such a suspension and the high gravity material toward the bottom. Two types of agitation of the water are presently used in Michigan — pulsating air and mechanical pulsing via a rubber diaphragm. The success of the process is, of course, based upon the assumption that most of the deleterious particles will be of lower specific gravity. Their removal will then provide a superior product.

Fig. 2 shows determination of deleterious content from carefully selected samples of one jig installation before and after processing. Also shown is the average freeze-thaw durability factor in each case for nine concrete beams made using vacuum saturated aggregates in 5½-sack air-entrained concrete. ASTM Method C 291, "Resistance of Concrete Specimens to Rapid Freezing in Air and Thawing in Water," was used and detailed procedures are substantially as reported in a previous paper.[2] The figure shows some reduction in deleterious content is found and consequent improvement in durability factor, but neither is sufficient to recommend this process. Jig processing increased the bulk dry specific gravity from 2.58 to 2.66 and reduced the absorption from 1.9 to 1.5. It is generally recognized that a jig operation is most successful where considerable disparity in specific gravity occurs between the acceptable and unacceptable gravel. A closer "cut" than provided by jigging is necessary to provide first class gravel from this deposit.

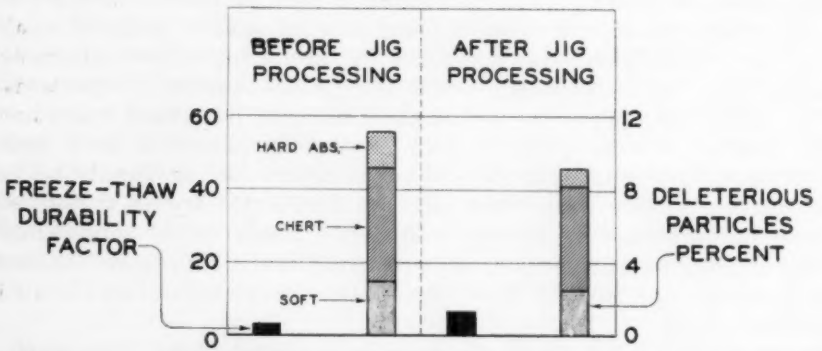

Fig. 2—Freeze-thaw durability factor and deleterious particles content before and after jig processing at one commercial installation

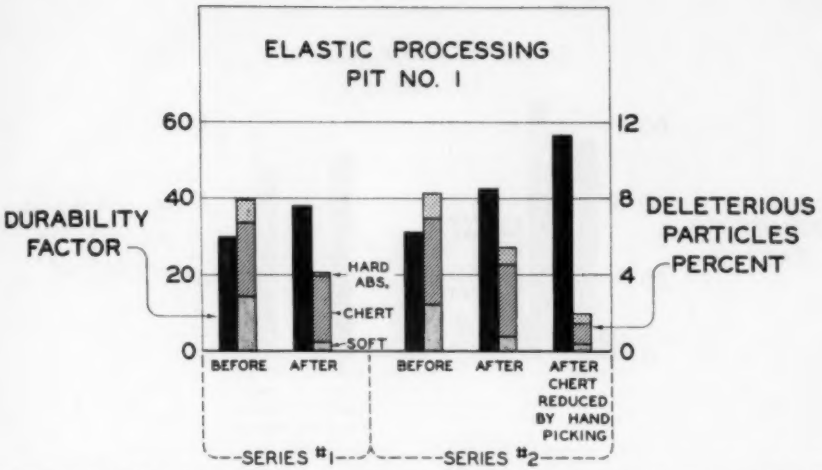

Fig. 3—Results of elastic fractionation or "bounce" process at one gravel pit. Note improved durability resulting from subsequently reducing chert content by hand picking

It should be emphasized that broad conclusions regarding the jig process should not be drawn from experience with this one installation. The numerous jigs around the country[5] attest to the fact that this process may be well adapted to the problems of some areas.

Elastic fractionation

This method of gravel beneficiation is based on the assumption that good and inferior quality particles may be separated by allowing them to fall on an inclined hardened steel plate. The hard, elastic particles will rebound a considerable distance, whereas the soft, friable particles may ricochet only a short distance or may actually disintegrate when striking the plate. Bins located at suitable distances from the plate will receive the particles thus "fractionated." Obviously, particle shape will affect the rebound and the process is best adapted to rounded particles.

Fig. 3 indicates results obtained from two series of tests at a deposit using this process. There was reasonable success in reducing soft particles and hard absorbent content with consequent improvement in freeze-thaw durability, but the process does not appreciably reduce the chert content. To confirm that chert removal would improve durability, sufficient of the elastic processed gravel was hand picked in the laboratory to reduce the chert content so as to provide the required amount for freeze-thaw determination in concrete. Note that this treatment substantially improved durability and confirmed that the important deficiency of the process at this deposit was its inability to reject chert.

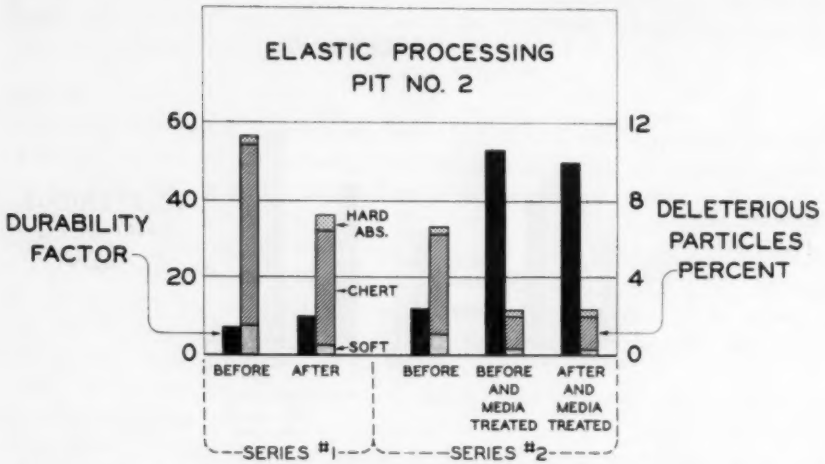

Fig. 4—Results of elastic fractionation of another gravel; also heavy media treated in laboratory, resulting in increased durability

Fig. 4 shows the results of two series of elastic fractionation tests at another pit. In this case, the over-all durability of the gravel was much less satisfactory. To give further insight into the method, the Series 2 tests included heavy media treatment of the gravel both before and after elastic fractionation. In this case, a laboratory media separation was made at a specific gravity of 2.55. Media treatment substantially improved the durability and reduced the deleterious content. No appreciable difference resulted from media treatment whether before or after elastic fractionation.

Fig. 5 shows similar data for a third deposit, using elastic fractionation. These tests again showed favorable results from laboratory heavy media treatment of the gravel at a specific gravity of 2.55. The results are particularly interesting since a heavy media plant was later installed at this deposit; results obtained from the commercially processed gravel using media treatment combined with elastic fractionation are discussed below.

Michigan experience with the bounce process is limited and the results do not warrant over-all conclusions. The pits in which this process was used were not more than 15 miles apart.

Cage mill

The cage mill is used in some applications for crushing, but is being tried in one Michigan installation for beneficiation. The mill consists of a single wheel, about 3 ft in diameter, rotating on a horizontal shaft, all enclosed within a strong steel enclosure. Mounted on the periphery of the wheel, equally spaced around the circumference, are 12 round

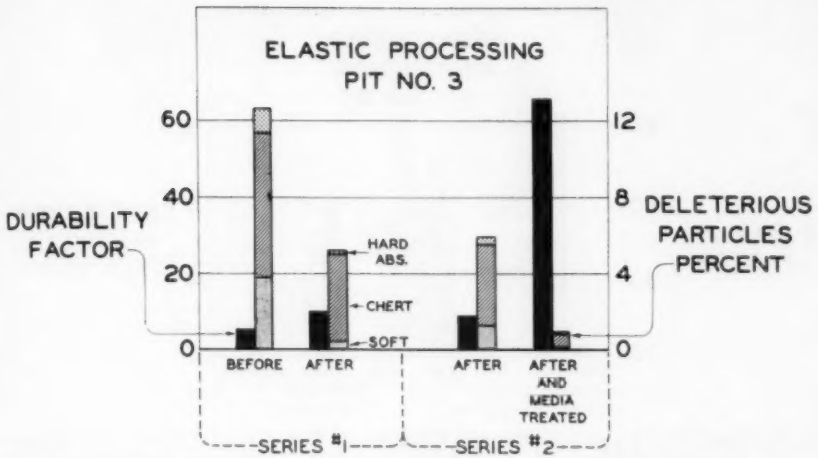

Fig. 5—"Bounce" processing for a third source with laboratory heavy media treatment greatly improving durability. (See Fig. 12 for results of commercial HMS installation combined with bounce process at this pit)

steel bars approximately 4½-in. in diameter with axes parallel to the main axis of the machine. Gravel is fed into the center of the spinning cage, rotating about 580 rpm, and the softer fragments, when sifting through the openings between the bars, are broken up when struck or when impacting against the surrounding housing. The treated product drops out the bottom by gravity.

Fig. 6 shows the results before and after beneficiation by the cage mill. The results are not encouraging for this pit either from reduction of deleterious material or increase of durability factor. Apparently many of the deleterious particles in this deposit are so hard as to not

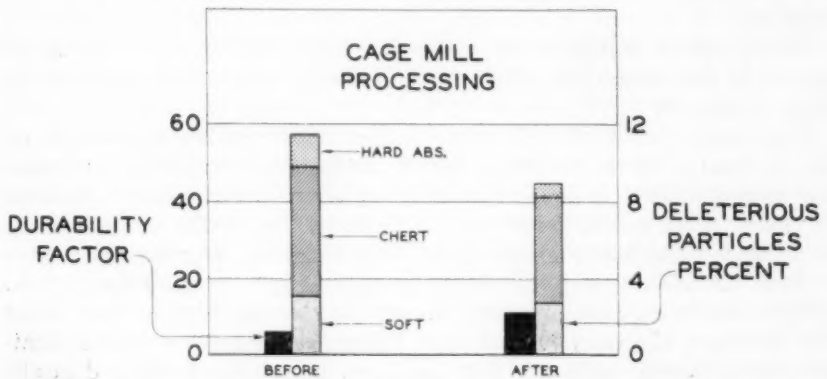

Fig. 6—Cage mill processing of a Michigan gravel; results show no improvement by this process for this deposit

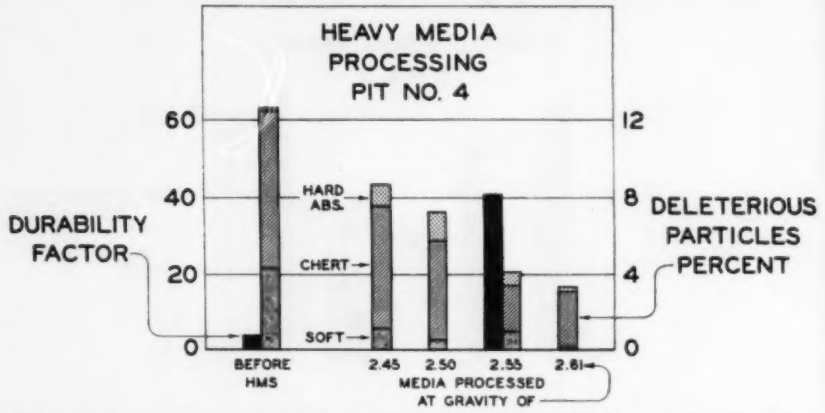

Fig. 7—Trial run of a gravel through commercial heavy media plant. Use of successively higher media gravities improved gravel correspondingly

be broken up by the cage mill. Again, more comprehensive data are needed to evaluate this process fully.

Heavy media separation

Heavy media separation accomplishes a specific gravity separation of gravel by floating off the lightweight material in a heavy liquid consisting of water suspension of finely ground magnetite and ferrosilicon. The process is successful for deposits in which the deleterious material is predominantly of lower specific gravity. This has generally, but not always, been found to be the case for Michigan gravels. Three types of separating devices using heavy media are now in the area: rotating drum, stationary vessel with reciprocating paddle, and cone type. All three require essentially the same auxiliary equipment for cleaning and magnetically recovering the magnetite and ferrosilicon.

Heavy media beneficiation plants are more numerous in the Michigan area than any of the other types, and data on these is consequently more extensive.

Experience has indicated that excellent information is derived on gravel from a source on which beneficiation is contemplated, by trucking approximately 50 tons to an existing plant and running it through at one or more media gravities. Fig. 7 shows the results of such a run at a commercial heavy media plant. It is observed that the deleterious content diminishes progressively as processing gravity is increased from 2.45 to 2.61. In no case, however, did the deleterious content quite meet the Michigan Highway Department 3 percent limit for exposed architectural concrete, although this has been achieved at other deposits. Durability of the gravel processed at 2.55 gravity shows a substantial increase over the untreated gravel.

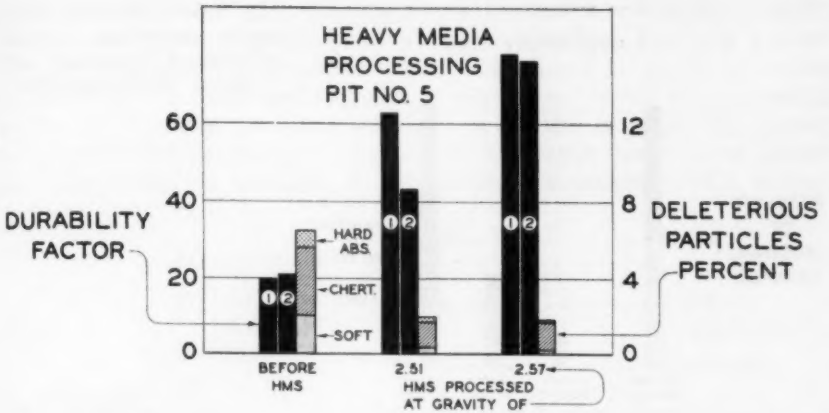

Fig. 8—Trial run of a gravel at a commercial HMS plant using two different media gravities; durability determined independently by two laboratories

Fig. 8 shows results of an experimental run at another pit. In this case, processing at a media gravity of either 2.51 or 2.57 provided material easily meeting Michigan's 3 percent deleterious requirement for high quality exposed concrete. In addition to the highway department laboratory tests, an opportunity was provided to conduct independent freeze-thaw concrete tests on the unprocessed and beneficiated gravel by the Joint Research Laboratories of the National Sand and Gravel Association and the National Ready Mixed Concrete Association at the University of Maryland. Concordance of durability results between the two laboratories was gratifying, although there was some variation for the material processed at 2.51 specific gravity.

Questions frequently arise as to the amount of material discarded during operation of a heavy media plant. Trial runs at a commercial plant have not been conducive to accurate determinations, but laboratory separations can readily provide answers. Fig. 9 and 10 present laboratory data acquired in contemplating HMS for a pit. These results are not unusual for commercial production; actual discard appears to be in the 15-20 percent range. With an operating gravity of as high as 2.60, roughly one-half of the 20 percent discard would be considered "satisfactory" gravel. Reference to the composition of the "sink" material indicates an extremely low deleterious content and such a high media gravity is unnecessary. A gravity somewhere between 2.50 and 2.60 would appear to be optimum for providing a high quality gravel without undue waste at this pit.

Favorable results for HMS for other Michigan installations have been reported elsewhere.[6] However, there are always exceptions, and one recent heavy media plant has proved relatively unsuccessful due to

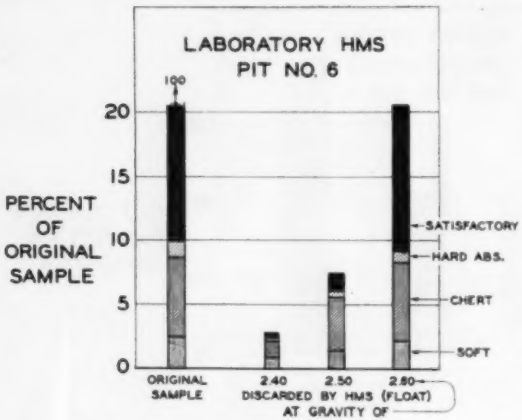

Fig. 9 — Laboratory heavy media separation; amount of discard increased with higher media gravities

presence of heavyweight iron claystones in the deposit. These were apparently not discovered in preliminary investigations of the newly developed pit and were not removed by heavy media processing. An insight into the effect of these iron claystones on concrete durability is provided by Fig. 11 where the individual durability factors of each of the nine concrete test beams before and after processing is shown. The durability before processing was uniformly low with an average of 2 whereas the durability after HMS provided an average of 33, but individual beams were extremely variable with a low of 1 and high of 82. Examination of the failed beams after breaking in flexure indicated that this variability was due to random location of the very destructive claystones. Interpretation of the freeze-thaw durability of this gravel is aided as much by observing the beam to beam variation as by the average durability. The iron claystone content of the beneficiated gravel was only 1.7 percent.

Lastly, Fig. 12 presents data from an interesting installation where the gravel is heavy media treated following elastic fractionation. The bounce process was not commercially feasible at this deposit because,

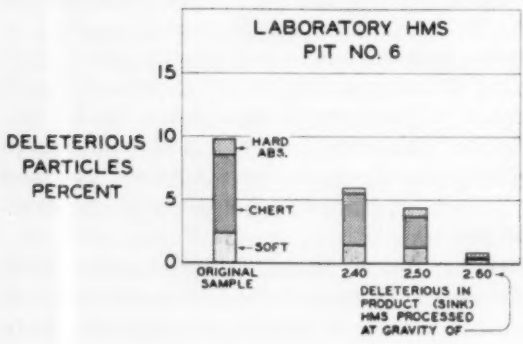

Fig. 10—Character of beneficiated product (same test as Fig. 9); very low deleterious content found using media gravity of 2.60

as indicated earlier (Fig. 5), the chert content was not appreciably reduced and a heavy media plant was later installed. The results from the latter commercial production are favorable both from reduction of deleterious content and increase of durability factor. By increasing media gravity, this plant has also been successful in producing gravel with less than 3 percent total deleterious. The latter beneficiated gravel has been used, for example, in fabricating prestressed bridge beams.

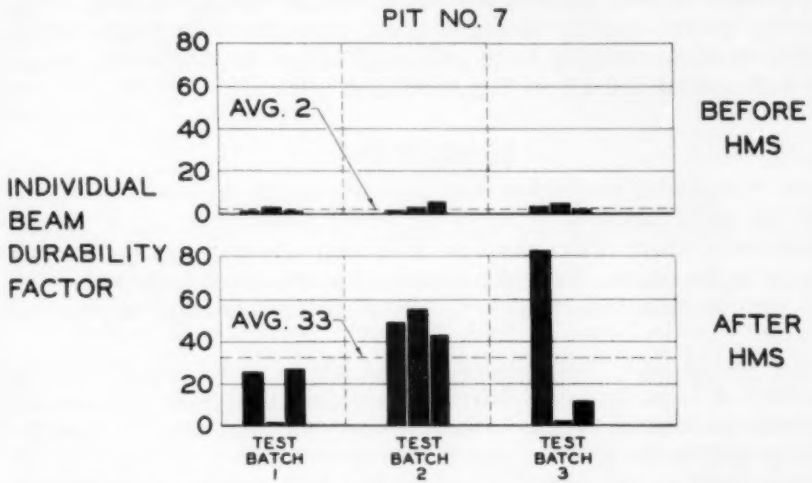

Fig. 11—Individual beam durability factors before and after processing at a commercial HMS plant. High gravity iron claystones were not removed by this HMS treatment and provided gravel of extremely variable durability

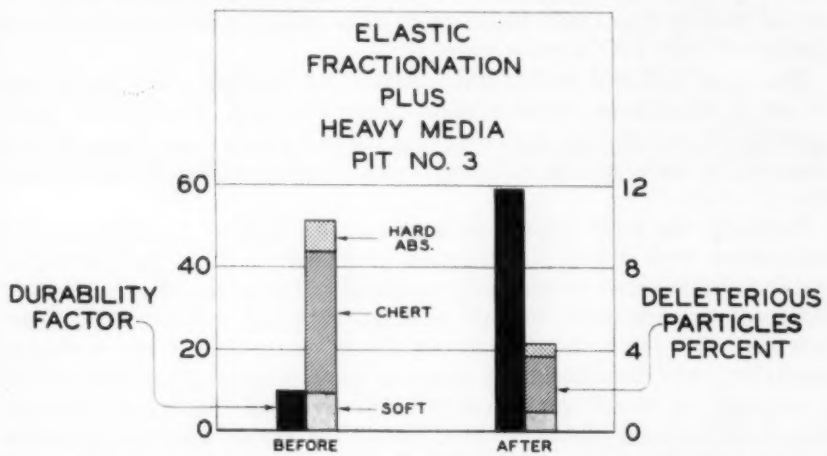

Fig. 12—Commercial installation using elastic fractionation followed by heavy media separation, thus providing good gravel (same pit as Fig. 5)

GENERAL EXPERIENCE WITH BENEFICIATION PLANTS

Routine day-to-day acceptance testing of aggregate from these beneficiation plants proceeds exactly in the same manner as at any gravel plant subject to highway department inspection—gradation analyses and deleterious determinations at frequent intervals with occasional check samples being sent to the central laboratory. The gravel operator operates his plant in a manner of his own choosing so as to maintain the product within specification limits. Special testing techniques involving specific gravity determinations, particularly at heavy media plants, have occasionally been proposed. So far, no compelling reason for such special procedures has developed.

CONCLUSIONS

An enthusiastic and resourceful gravel industry in southern Michigan has been particularly active in recent years in applying special processes to remove deleterious particles from the glacial gravels which abound in the region. Users of concrete in the area, both large and small, had become dissatisfied and had demanded an upgrading of concrete quality.

Best results have been achieved using heavy media separation, but presence of high density deleterious particles in at least one deposit indicates limitations of this process when both high and low specific gravity deleterious particles are in the same gravel.

Experience so far with the jig method has proved less successful since it does not appear to make as close discrimination in specific gravity as these gravels demand. For example, in the case of heavy media processing, it has been observed that substantial changes in gravel quality may result for some of these deposits by making a media change of only 0.05 specific gravity.

The cage mill and elastic fractionation, or "bounce" method, appear to not give sufficient consideration to the fact that, for some of these gravels, the deleterious particles may be quite hard and resilient and thus not be removed by either of these processes. This is particularly true of chert.

Probably the most important lesson to be learned from Michigan's experience with these beneficiation processes is that the particular method chosen must be carefully matched to the needs of each deposit. Different gravels may demand entirely different processes of beneficiation. The most satisfactory means of determining the technical feasibility—and this has been done on numerous occasions—is to run a truckload or two of the gravel to be investigated through an existing beneficiation plant. Generous size lots of the gravel are thus provided for whatever tests may be desired. Further, there is little question of the validity of the run when put through a full scale plant, as com-

pared to laboratory separations of smaller samples. The gravel producer must, of course, consider the economics of the installation. The original investment will be substantial and he certainly should consider market potential, means of disposal of waste product, and such matters.

Finally, establishment of the several beneficiation plants is bound to have a salutary influence on over-all concrete quality in the area. Despite greatly increased production of aggregates in recent years, relaxation of quality standards has been unnecessary. Beneficiation has enabled some gravel producers not only to survive competition, but to be able to furnish premium aggregates complying with rigid specifications for architectural concrete where severe exposure conditions are anticipated.

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Title No. 57-41

The Structural Membrane

By KOLBJORN SAETHER

Even though ideal from a structural point of view, elastic membranes have been almost excluded from the structural field due to the complexity of mathematical work involved. Structural membranes, however, which are close to identical in appearance and structural behavior, permit the use of only elementary mathematics for defining the shape and analyzing the stresses within its surface. The basic theory of funicular shapes and the transformation of these into structural membranes is shown.

Savings on materials typical for all thin shell structures, together with ease of design and construction, are some of the advantages promised by structural membranes.

■ IN THE STRUCTURAL FIELD, where man is mastering larger forces than in almost any other technical branch, applications of the elastic membrane have been limited. Except for steel cables in funicular structures, few available structural materials come even close to having the qualities of an elastic membrane. In the following it will be shown how a stiff material such as concrete may be used with the same efficiency and elegance to carry floor and roof loads as the sail is used to contain the driving force of the wind.

If a complete stress reversal is assumed the shape of an elastic membrane under tension would be ideally suited for a concrete structure under compression. This idea in itself is not new and a vast amount of literature exists on this subject. The problem in the past has been that the funicular shape (the shape of an elastic membrane) is so complex as to obstruct easy assignment of engineering dimensions, stresses, loads, and other necessary data.

In this paper, however, the funicular shape has been very closely approximated by using three known geometrical shapes, all of them readily defined and each one of them already separately used in structural design. These shapes are the parabolic elliptical dome, the hyperbolic paraboloid, and the logarithmic elliptical funnel.

A structural membrane is defined as a combination of two or more of these three geometrical shapes fused together into one continuous surface.

To understand the behavior of a structural membrane it is necessary to examine what takes place when the shape of an elastic membrane

is used in a concrete structure under compression. After this it will be shown how the shape of an elastic membrane can be approximated and simplified by the structural membrane.

CONVERTING AN ELASTIC MEMBRANE UNDER TENSION TO A SIMILAR FUNICULAR STRUCTURE UNDER COMPRESSION

By comparing an elastic membrane under tension with the corresponding structural shape under compression a number of conclusions about the structural conditions in the latter shape may be made. Under distributed loads w normal to the membrane, the elastic membrane will deflect in such a manner that the internal stresses S, all acting in the direction of the deflected surface, will have their resultants r in perfect equilibrium with the loads (Fig. 1).

This condition may be expressed as follows:

$$S_{xy} = F_{xy}(w) = F_{xy}(r) \quad \text{.....(1)}$$

Due to the nature of the elastic membrane the stresses S are all tensile stresses.

If all signs in Eq. (1) are changed, providing the shape is maintained, the equilibrium is not disturbed (Fig. 2). This analogous condition may be expressed mathematically:

$$-S_{xy} = F_{xy}(-w) = F_{xy}(-r) \quad \text{.....(2)}$$

Whereas Eq. (1) indicates a statical condition that is always satisfied in an elastic membrane, a quick check of Eq. (2) shows that these analogous conditions could not all be handled by the same membrane.

TABLE 1 — TRANSFORMATION OF PROPERTIES FROM ELASTIC TO STRUCTURAL MEMBRANE

Elastic membrane [Eq. (1)]	Funicular Structural Shape [Eq. (2)]
Loads on the elastic membrane model to be selected so that they are opposite to the actual loads on the structure	The loads are assumed known as those for which the structure has to be designed (Generally total dead and live load)
Certain tensional edge loads required to stress the membrane and to maintain the proper location of the edges	Complete reversal of all edge loads (Often handled by assuming beam-action along the edges of the structure)
Tension throughout	Compression throughout
Flexible surface	Rigid surface
All bending moments equal zero	All bending moments equal zero
Stabile equilibrium	Labile equilibrium
Shape is automatically that of the deflected membrane	Shape made identical to that of the deflected membrane

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Mr. Saether's experience includes thin shell designs, precast and slip-form construction problems, drilled-in caissons, lift-slab, blast resistant design, and development of a direct design method for prestressed concrete.

Table 1 shows how the properties known to exist in the elastic membrane are transformed in the analogy.

The importance of the analogy given by the transformation from Eq. (1) to Eq. (2) is easily understood by checking each item in the second column of Table 1. Item for item they show the ideal conditions for a concrete shell. The high compressive stresses located within the surface itself permit full use of the compressive strength of the concrete. These stresses are furthermore nearly uniform throughout the shell without concentrated loads or line loads. By definition the structure is initially free of any bending moments. Only secondary moments are introduced by changes in the live load and by edge loads.

Having examined what happens when the exact shape of an elastic membrane is used in a structure under compression, the next step is to define this shape. Instead of doing this mathematically or experimentally, an entirely different approach is shown.

Through observation of elastic membrane models it has been possible in a number of cases to approximate the shape of the elastic membrane with the three specified shapes of a structural membrane. While there is no apparent limit to the number of different elastic membranes which can be approximated in this manner, this paper is confined to the description of three basic types of structural membranes.

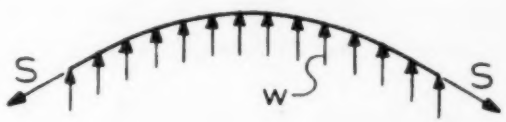

Fig. 1—Equilibrium of loads and stresses in elastic membrane

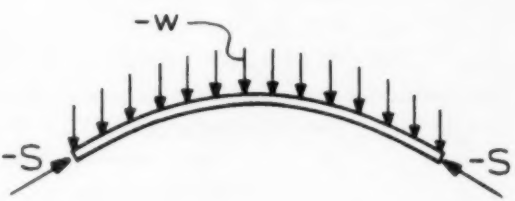

Fig. 2—Equilibrium of loads and stresses in structural membrane

EXAMPLE 1 — AN ELASTIC MEMBRANE SPANNING A ROUND OPENING

The first elastic membrane is created by a rubber sheet placed over a round opening and inflated (Fig. 3a). The resulting shape is found to be a dome (Fig. 3b).

The corresponding structural membrane consists of a circular parabolic dome, a special case of the elliptical parabolic dome. The results in itself being of little importance as this is a well known and commonly used structure, it should be noted that this structural membrane approximates the elastic membrane more completely than does the often used spherical dome.

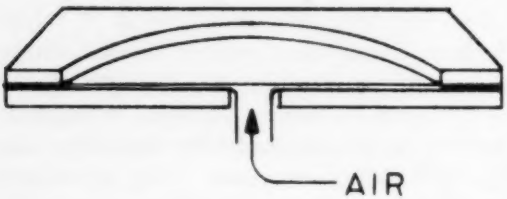

Fig. 3a—Apparatus for creating elastic membrane

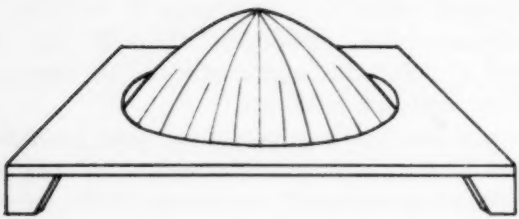

Fig. 3b—Circular parabolic dome

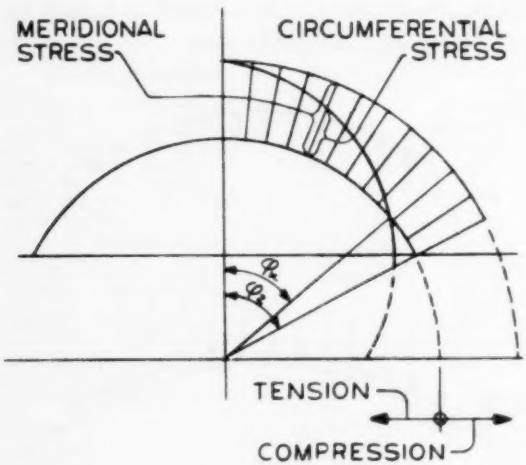

Fig. 4—Stresses in a spherical dome

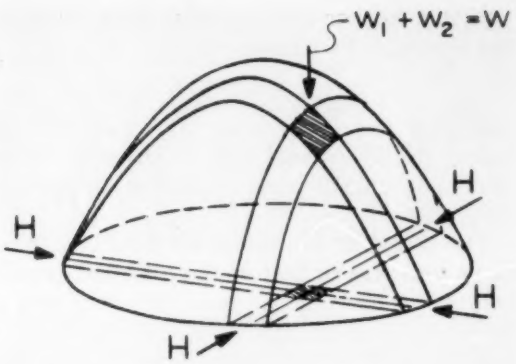

Fig. 5a—Surface generation in parabolic dome

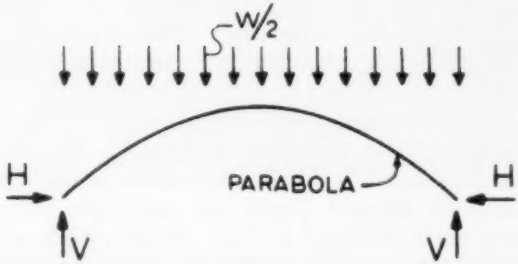

Fig. 5b — Thrust conditions in parabolic dome

The reason for this is apparent by noting that on a spherical dome the circumferential stresses decrease to zero along a definite line below which the dome is in tension in the circumferential direction (Fig. 4). This stress reversal does not exist in the elastic membrane as discussed in connection with Table 1.

The elliptical parabolic dome, on the other hand, is close to identical to the deflected elastic membrane. If a uniformly stretched elastic membrane is deformed only slightly, the stresses in the sheet will undergo only small changes and it may be assumed that the horizontal thrust, H, at any point of the sheet will remain close to constant throughout the membrane.

The dome may now be assumed subdivided into two series of parallel running arches intersecting at 90 deg (Fig. 5a). Each element of the dome is covered twice by these arches so that the total unit load on the dome, W, may be assumed divided into two parts. W_1 and W_2, the first part carried by one set of parallel running arches, the other part carried by the intersecting set of arches. The following equation will therefore apply to any element on the dome:

$$W_1 + W_2 = W \quad \text{(3)}$$

If it is further assumed that the total load is equally divided between the two sets of arches:

$$W_1 = W_2 = W/2 \quad (4)$$

and with the assumption that the thrust is constant throughout, the shape of these arches are completely determined, since the only arch that will satisfy these conditions is an arch with a parabolic shape (Fig. 5b).

The shape of the deflected elastic membrane is thereby determined to be identical to that of the elliptical paraboloid. This surface may be created by moving one parabola along and normal to another parabola. In a rotation symmetrical case as this is, the two parabolas are identical and the same surface could have been created by rotating the parabola about its axis. The parabolic dome thus discovered is not new; it has been used for some time. The important thing is that it indicates a complete correspondence between the elastic membrane and the structural membrane.

Even though used in a number of structures the parabolic dome permits little flexibility in design. The difficulty of combining more units within the same structure and its requirement of continuous circular support limits its usefulness. The following two examples are quite different in that respect. They are limited by straight, horizontal lines which make it possible to combine any number of units within the same project.

EXAMPLE 2 — SQUARE ELASTIC MEMBRANE TRANSLATED INTO A STRUCTURAL MEMBRANE

The next elastic membrane was built with a rubber sheet over a square opening. To account for the effect of a uniform dead and live load, air pressure was introduced under the membrane. A downward acting concentrated load was applied in the center as shown in Fig. 6, to correspond to the column reaction.

A study of the resulting shape shows a close to rotation symmetrical funnel shape in the vicinity of the column. Towards the corners the membrane has every characteristic of a saddle shape, similar to the hyperbolic paraboloid. A close investigation will further show that the transition from one shape into the other takes place along diagonal junction lines through the midpoint of each side, denoted with a in Fig. 7. The corresponding structural membrane consists of a logarithmic circular conoid with the four corners consisting of hyperbolic paraboloids.

The compressive stresses located within the surface itself permit full use of the compressive strength of the concrete. As in the elastic membrane, these stresses are furthermore nearly uniform throughout the shell without concentrated loads or line loads. By definition the

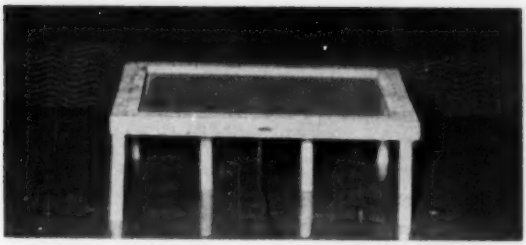

Fig. 6a—Elastic membrane uninflated

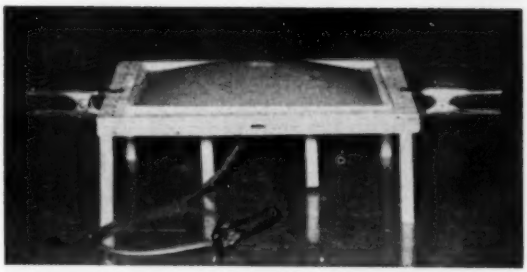

Fig. 6b—Elastic membrane inflated

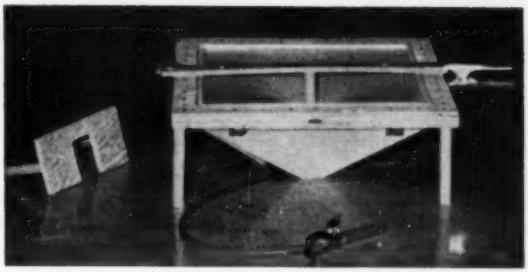

Fig. 6c—Elastic membrane inflated with downward load applied

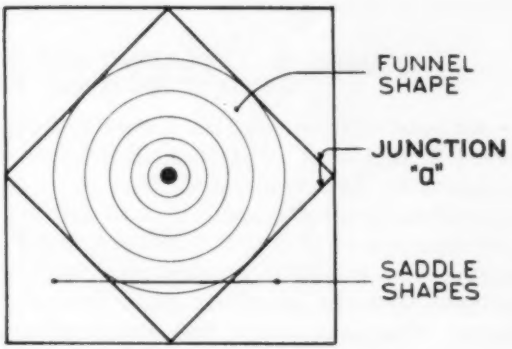

Fig. 7—Shape transition in structural membrane

Fig. 8a—One-story one-column house

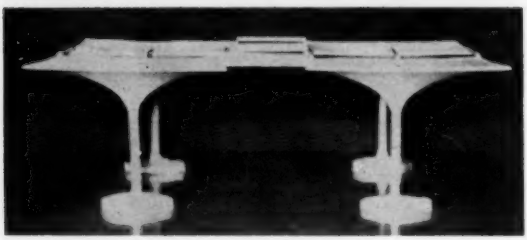

Fig. 8b—Four-column office

structure is initially free of any bending moments. Only secondary moments are introduced by changes in the live load and by removal of the edge loads.

The problem of physically measuring or analyzing a funicular shape is eliminated, and yet the shape offers many of the structural advantages of the funicular shape. This shape can be constructed virtually without beams, and all the concrete, except at the base of the conoid at the column, may be of constant thickness. The models in Fig. 8 show a one-story, one-column home and a four-column office layout with intersecting strips to provide light and ventilation.

The fact that this conoid and hyperbolic paraboloid may be fused into one continuous uninterrupted shape will be shown in detail in Example 3, along with mathematical data establishing and analyzing the use of these shapes.

EXAMPLE 3 — CREATING AND DEFINING A MORE VARIED STRUCTURAL MEMBRANE

An elastic sheet is inflated over a square opening with four downward acting loads. The funicular shape supported by evenly spaced columns is thereby created (Fig. 9a). Air pressure underneath the sheet represents the uniform load on the structure. A thin layer of concrete is cast on top of this sheet, and Fig. 9b presents the resulting shape. The conoid is quickly recognized as the area adjacent to the column with the parabolic dome located diagonally between the columns. These areas have been indicated as Areas 1 and 3, respectively,

Fig. 9a—Casting funicular shape

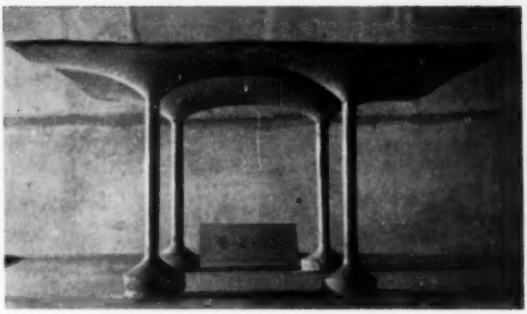

Fig. 9b—Resulting casting

in Fig. 10. The hyperbolic paraboloid shows up as transition sections between the other two areas and is located half way between the columns along the column lines, denoted as Area 2 in Fig. 10. A close observation of the model further reveals that the transition from one

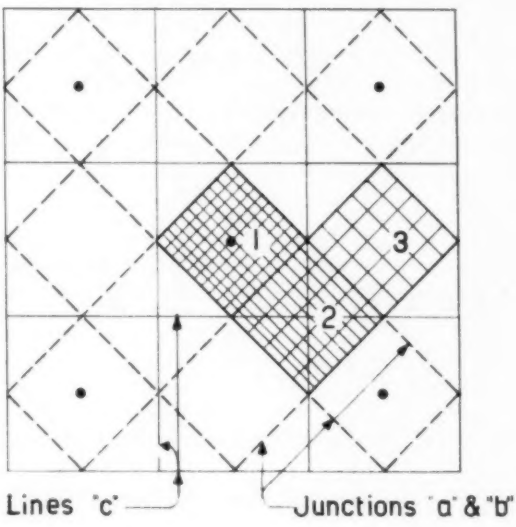

Fig. 10—Plan of structural membrane showing relative location of funnel (1), saddle (2), and dome (3)

area into the adjacent takes place along straight vertical sections, junctions a and b as shown in the same figure.

In Fig. 11 the same three areas are shown in larger scale. In each area a coordinate system has been introduced. Fig. 12 shows a section through the surface. By selecting the proper parameters the equations for these three areas may be expressed as follows:

Area 1 $$X_1^2 + Y_1^2 = F(Z) \quad (5)$$

Area 1a $$X_1^2 + Y_1^2 = -\frac{L^2}{4}\left(1 - \frac{Z}{h}\right) \quad (5a)$$

Area 2 $$X_2 Y_2 = \frac{L^2}{8}\left(\frac{Z}{h}\right) \quad (6)$$

Area 3 $$X_3^2 + Y_3^2 = \frac{L^2}{4}\left(1 - \frac{Z}{h}\right) \quad (7)$$

The three coordinate systems have been used simply for the purpose of keeping the above equations in such shape that they may readily be recognized. The ordinate Z is the same in all cases, the ordinates X and Y differ only in that they have been translated from Area 1

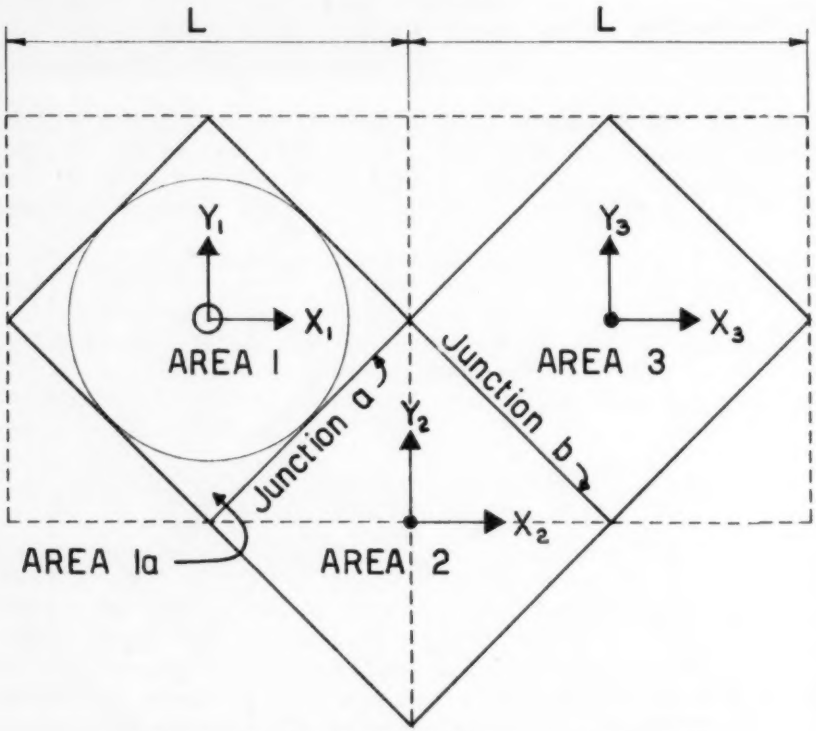

Fig. 11—Expanded view of three areas in Fig. 10

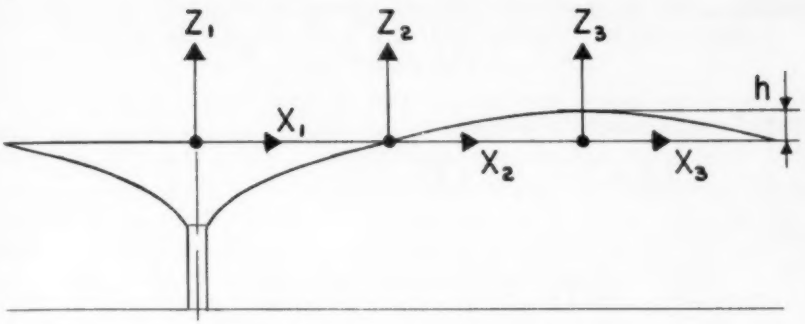

Fig. 12—Section through surface

with the amounts $X = +L/2$ and $Y = -L/2$ for Area 2, and $X = +L$ and $Y = 0$ for Area 3, respectively.

Area 1 in Fig. 11 has been divided into two parts, one circular Area 1 and four triangular segments denoted as Areas 1a. The above expressions for these areas show that the two Areas 1 and 1a are identical, with 1a as a special case of 1. As it will be shown in the following, Eq. (5) may take several specific shapes. The only limitations are that $F(Z)$ and $F'(Z)$ must coincide with those of Eq. (5a) for values of $X^2_1 + Y^2_1 = L^2/8$, that is: the location of Surface 1 and its slope must be identical with that of Surface 1a along the circle with radius $L/\sqrt{8}$. That these requirements can be satisfied is evident due to the similarity of the two equations. That the other equations describe one continuous shape may not be quite so evident; this will be proven in the following:

Since Eq. (5a) and Eq. (7) are identical but opposite and since Eq. (6) is equal but opposite in the two quadrants limited by the junction lines a and b, respectively, it is enough to show that Eq. (6) and (7) satisfy the continuity requirements. To establish the location of the surface along the junction the equation for the vertical surface intersecting the structural membrane along the junction line a must be written out. Expressed in terms of coordinate System 2 it is:

$$X_2 + Y_2 = \frac{L}{2} \text{ or } X_2 = \frac{L}{2} - Y_2$$

This value for X_2 inserted in Eq. (6) gives the following expression for the junction line:

$$\left(\frac{L}{2} - Y_2\right) Y_2 = \frac{L^2}{8}\left(\frac{Z}{h}\right) \quad \text{or} \quad \frac{L}{2} Y_2 - Y^2_2 = \frac{L^2}{8}\left(\frac{Z}{h}\right)$$

The corresponding vertical surface as expressed in terms of X_3 and Y_3 is:

$$X_3 + Y_3 = -\frac{L}{2} \quad \text{or} \quad X_3 = -\frac{L}{2} - Y_3$$

This value for X_3 inserted in Eq. (7) gives:

$$\left(\frac{L}{2} + Y_3\right)^2 + Y_3 = \frac{L^2}{4}\left(1 - \frac{Z}{h}\right)$$

Since $Y_3 = Y_2 - L/2$ it follows:

$$\left(\frac{L}{2} + Y_2 - \frac{L}{2}\right) + \left(Y_2 - \frac{L}{2}\right)^2 = \frac{L^2}{4}\left(1 - \frac{Z}{h}\right)$$

and

$$2\,Y_2^2 - Y_2 L + \frac{L^2}{4} = \frac{L^2}{4} - \frac{L^2}{4}\frac{Z}{h}$$

With $L^2/4$ eliminated and the resulting expression divided by 2 and changing signs throughout it is found:

$$\frac{L}{2}Y_2 - Y_2^2 = \frac{L^2}{8}\left(\frac{Z}{h}\right)$$

This is, however, the same curve as established by using Eq. (6) and proves that the two surfaces are at the same elevation at any point along the junction line.

To prove that there is a smooth transition from one surface to the adjacent one, it is enough to prove that the surface is continuous and of constant slope in any one direction outside the direction of the junction line itself. The simplest expressions are obtained by establishing the slope of the surface in the direction parallel to the X-axis. (The Y-axis could have been chosen with the same convenience.) The slope of Area 2 in the direction of the X-axis is expressed by taking the partial derivative of Z in this direction. From Eq. (6) it is:

$$\frac{\partial Z}{\partial X} = \frac{8h}{L^2}\,Y_2$$

and from Eq. (7) the equivalent value is:

$$\frac{\partial Z}{\partial X} = \frac{4h}{L^2}\,2X_3 = \frac{8h}{L^2}\left(-\frac{L}{2} - Y_3\right) = \frac{8h}{L^2}\left(-\frac{L}{2} - Y_2 + \frac{L}{2}\right) = \frac{8h}{L^2}\,(-Y_2)$$

This last expression is equal but opposite to that established by using Eq. (6) and proves that the surface has a constant slope across the junction lines.

The basic geometry of the structural membrane is thus established. Just as there is a series of possible variations to the regular, symmetrical hyperbolic paraboloid the same applies to the structural membrane. Structural layouts where the three basic shapes of the conoid, the hyperbolic paraboloid, and the parabolic dome have been changed from having a regular square outline as shown here to a pattern of triangular and hexagon shapes; all triangular shapes; and all rectangular or all diamond shapes have been successfully combined to cover a long variety of column arrangements.

STRESSES IN STRUCTURAL MEMBRANE SUPPORTED ON EVENLY SPACED COLUMNS DUE TO UNIFORMLY DISTRIBUTED VERTICAL LOAD

Even though partial loading may control the design of some thin shell structures, the great majority of shells constructed till now have been based upon a design considering the uniform total load only. This is partly due to the practical limitation of our knowledge about shell structures, but also due to the fact that most shells, and especially those of double curvature, have remarkable ability to redistribute the load with the effect that the total load governs the design. In this paper only the total, uniformly distributed load will be considered.

If the unit horizontal thrust were known for any one section of a shell the actual stresses within the surface itself could be readily determined by the following expression:

$$F = H \cos w / \cos u \quad \text{.....(8)}$$

where H is the unit horizontal thrust, u, the angle of slope of the shell surface in a direction normal to the horizontal projection of the section, and w is the slope of the section itself. The unit stress F would not necessarily be exactly acting normal to the section, but for small values of the angle w the discrepancy would be small and could safely be ignored in the design. For larger values of w an adjustment may be required. Eq. (8) is general and applicable to any type of thin shell structure, and the stress investigation is thereby reduced to the determination of the required horizontal thrust.

As shown in Example 1 the parabolic dome may be assumed to be divided into two series of intersecting arches by vertical planes running parallel with the junction lines. These arches so established will all be identical, a fact readily verified by the knowledge that a parabolic dome may be described by translating one parabola along and normal to a similar parabola. Since the height of these parabolas is $h/2$ and the span of the same parabolas is equal to $L/\sqrt{2}$ and further since the load, due to absolute symmetry of the shell in this area, may be assumed divided equally between the two sets of arches, the required thrust throughout and along the edges of the dome area is determined to be:

$$H_1 = \frac{w L^2}{16 h} \quad \text{.....(9)}$$

At the junction lines b the parabolas from the dome runs into similar parabolas in the hyperbolic paraboloid, the latter, however, having reversed curvature. If the thrust is assumed to remain unchanged these parabolas will have a downward vertical resultant along the entire length of the arch of the magnitude $w/2$. The intersecting set of arches within the hyperbolic parabolic area will therefore in addi-

tion to the load w have to carry the load $w/2$ from the first set of arches. The later group is composed of arches with concave curved parabolas similar to those in the dome area. The required thrust to carry the total load of $3/2\,w$ is therefore established as:

$$H_2 = \frac{3\,wL^2}{16\,h} \qquad (10)$$

The compatability requirements between parallel running arches with different thrust values has been found to be of no serious consequence. It may be stated here, without going further into this problem, that instead of a sudden change in thrust from one group to the adjacent, it may be assumed that a gradual transition will take place. Minor moments and shears normal to the surface of the shell do accordingly exist along the junction lines b.

The established thrust of [Eq. (10)] carries all load down to the lines a at the junction with Area 1a.

The triangular Areas 1a limited by the lines a and bordering Area 1 form somewhat of a transition surface as far as the stresses are concerned. These stresses are not as readily determined as in the other parts of the shell. It may be concluded, however, that due to its similarity with the elastic membrane, the stresses in these areas will be governed by the same basic stress pattern as the rest of the shell. It is important to note, however, that due to the convex curvature of both sets of arches in these areas a certain amount of shear exists. The complete picture of what happens in the Area 1a can only be arrived at after the Area 1 has been investigated.

It was stated earlier that Area 1 could take a number of shapes. According to Eq. (5) they all must be rotation symmetrical, however, and the slope of the surface along the junction with the Areas 1a and 2 must coincide with those of the latter areas. This establishes the required thrust along the border of Area 1, this border being a circle with a radius of $R = L/\sqrt{8}$. The slope angle along this border is given from Eq. (6) for $X_2 = -L/4$ and $Y_2 = L/4$ as $\tan U = Z_2' = 8h/L^2$ $(Y_2 \cos \phi + X_2 \sin \phi)$ with $\phi = 135$ deg.
Since $\sin 135 \text{ deg} = 1/\sqrt{2}$ and $\cos 135 \text{ deg} = 1/\sqrt{2}$ it is:

$$Z_2' = -\frac{8h}{L^2}\left(\frac{L}{4\sqrt{2}} + \frac{L}{4\sqrt{2}}\right) = -\frac{\sqrt{8}\,h}{L}$$

The total vertical load to be carried by this circle may be established from a free body diagram separating Area 1 from the rest of the shell:

$$P_{x\,=\,L/\sqrt{8}} = 2wL^2 - \frac{\pi wL^2}{8}$$

Since the vertical component of the stress is equal to the horizontal

thrust H_3 times Z_2', the following expression is established for this thrust:

$$H_3 = \frac{wL^2\,(2 - \pi/8)}{2\pi h} \qquad (11)$$

This thrust, which also may be expressed as $H_3 = 0.255\ wL^2/h$ as compared with $H_2 = (3wL^2)/16h$ or $H_2 = 0.188\ wL^2/h$ along the junction lines a, explains the existence of shears in the Area 1a.

The last Area 1 may now be fully investigated. Several approaches are possible, the most logical one appears to be to assume that the established thrust of Eq. (11) is maintained throughout this area. Based upon this assumption the following differential equation may be derived:

$$Z' = \frac{\left(2 - \pi\left(\frac{X}{L}\right)^2\right)h}{(2 - \pi/8)\,X}$$

which gives as general solution:

$$Z = 1.244\ h\ \log_e\,(X/L) - 0.977\ h\ (X/L)^2 + C_1$$

Inserting the edge requirement that for $X = L/\sqrt{8};\ Z = -h/2$ the constant C_1 may be determined:

$C_1 = -h/2 - 1.244\,h \log_e\,(1/\sqrt{8}) + 0.977\,h\,(1/8) = 0.916\,h$ and it follows:

$$Z = 1.244\,h \log_e\,(X/L) - 0.977\,h\,(X/L)^2 + 0.916\,h \qquad (12)$$

This result indicates a logarithmic radial curve for the funnel surface. The extreme values of Z for X approaching zero may be disregarded since the shell in this vicinity will have fused into the column.

With the entire surface determined as also is the corresponding horizontal thrust, the actual stresses are readily determinable by means of Eq. (8).

EDGE LOADS ON THE STRUCTURAL MEMBRANE

This discussion is limited to square, structural membranes with straight horizontal edges. This divides the previously discussed surface into two types each with a side dimension of L as shown with dotted lines in Fig. 11. The one containing the column is referred to as "the column unit;" the other as "the dome unit." In the following a precast structure utilizing these two types of units is considered. In this case all areas adjacent to the edges are hyperbolic paraboloids. The problem of edge loads on these structural membranes is therefore similar to that of the hyperbolic paraboloid. A basic difference exists, however, in that in a conventionally designed hyperbolic paraboloid its support is so selected that only shears may be assumed to exist along its edges, whereas in the structural membrane

these edges are subjected to large normal forces in addition to the shears.

The following is a general approach to the solution of this problem. The horizontal edges of the structural membrane require two types of edge loads to maintain equilibrium within the shell surface, one tangential force U acting normal to the edges and one shear force S acting along the edges. The force U may be divided into two components, one component V vertical and one component H horizontal. Since the horizontal thrusts, H_1 and H_2, as earlier established, describe the entire horizontal load on the shell, these thrusts may be used to determine the horizontal edge load H. The stresses H_1 and H_2 may be found to act in two directions inclined 45 and 135 deg to the edges, and the horizontal edge load is therefore:

$$H = \frac{WL^2}{16\,h}(3 \sin 45 \text{ deg} + 1 \sin 135 \text{ deg})\ \tfrac{1}{2} = \frac{wL^2}{8\,h} \quad \ldots\ldots (13)$$

or also $H = 2\,H_1$.

This force is constant along the entire edge. The vertical edge load v is then determined as $H \tan u$, where u is the slope of the surface normal to the edge. This $\tan u$ varies linearly from zero at the corner to a maximum value $\tan u = 4h/L$ at the center of each side. The total vertical load on the unit is therefore:

$$W = 4LH\,(\tan u)_{max}\,1/2 = 4L\,\frac{wL^2}{8h}\,\frac{4h}{L}\,1/2 = wL^2$$

This force acts upward on the dome unit and downward on the column unit and checks with the over-all statical requirements of the structural membrane.

The shear s may similarly be established from the two thrust values H_1 and H_2. It is:

$$S = \frac{wL^2}{16h}(3 \cos 45 \text{ deg} + 1 \cos 135 \text{ deg})\ 1/2 = \frac{wL^2}{16h} \quad \ldots\ldots (14)$$

or also $S = H_1$.

This shear is constant throughout. On the dome unit it acts from the midpoint of each side toward the corners and in opposite direction on the column unit.

For the free edges of any larger structural membrane it is recommended that the vertical edge loads be transferred from the dome unit to the column unit by means of vertically placed edge beams. (A 10 ft square test unit 2 in. thick, however, showed negligible deflections of the edge when loaded to 300 lb along one unsupported edge.) Within the structure itself this same vertical edge load is automatically cancelled by the fact that each dome unit rests on the edges of the column-unit. The horizontal edge loads do not normally cancel out, however. Only if mechanical fastening devices were provided at the junction between the units a smaller or larger part of these horizontal forces would cancel

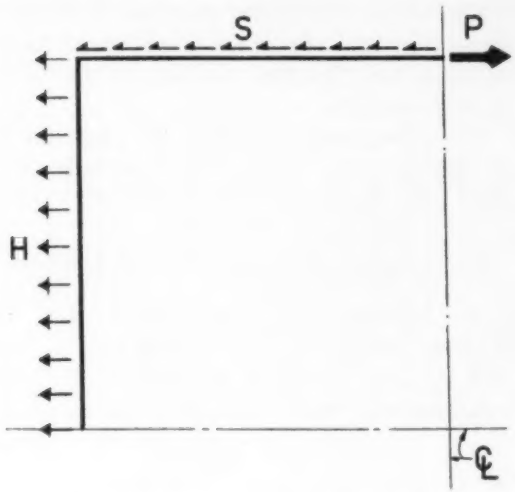

Fig. 13 — Horizontal edge loads in direction of *x*-axis. One-quarter of structural membrane column unit is shown

out within the structure. In the following no such devices will be assumed. These horizontal loads would set up tension in the structural membrane.

If prestressing wires are used to balance the horizontal loads, the magnitude of the required prestressing may be determined from the free body diagram in Fig. 13. Here one quarter of the column unit has been shown. Only the forces in the direction of the *X*-axis have been indicated. The forces in the *Y*-axis would be the same due to the symmetry of the unit. The diagram establishes the following values for the required prestressing force:

$$P_{reqd} = H\,L/2 = 2H_1\,L/2 = H_1L \text{ at the corners}$$

and

$$P^{max}_{reqd} = H_1L + S\,L/2 = 3/2\ H_1L \text{ at the center line of the column unit.}$$

The corresponding values for the dome unit are:

$$P^{max}_{reqd} = H_1L \text{ at the corners}$$

and

$$P_{reqd} = H_1L - S\,L/2 = 1/2\ H_1L \text{ at the center line of the dome unit}$$

Whereas this prestressing force satisfies all of the over-all statical requirements of the shell the combined effect of the edge loads and the prestressing force on the internal stresses in the structural membrane must be analyzed.

The following discussion presents a more qualitative than quantitative evaluation of this effect, any exact analysis not being available at the time this paper is written. In the author's opinion, however, this discussion comes close to describing the actual stress pattern within the shell.

Fig. 14 represents one quadrant of a unit. The combined prestressing force from the two sides is shown to have a diagonal resultant $P\sqrt{2}$ at the corner. The shears along the prestressing wire have been ignored in this diagram.

The curves shown within the shell are contour lines plotted at intervals of $h/10$. Within the hyperbolic paraboloidal part of the unit these are all hyperbolas of increasing focal distances as they occur further away from the corner.

These contour lines may now be assumed to be separate, rigid arches each carrying an axial load p. Due to the curvature of the axis of these arches horizontal reactions h are created. The approximate distribution of these reactions is indicated in Fig. 14. If $\Sigma p = P$, it is readily proven that the total resultant of these forces h will be in equilibrium with the diagonal load $P\sqrt{2}$. The presence of shears and forces normal to

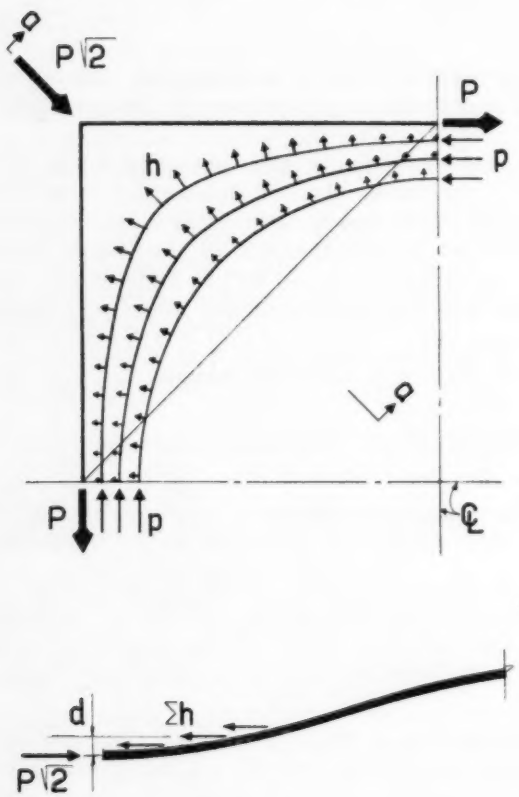

Fig. 14 — Stresses due to prestressing. One-quarter of structural membrane unit is shown

the horizontal arches will modify but not basically change this stress pattern.

Since these arches by definition are horizontal, the compressive forces p have no vertical component. Accordingly the basic equilibrium of the shell in the vertical direction has not been disturbed.

The horizontal edge loads H for a dome unit are shown in a similar diagram in Fig. 15. The contour lines are the same horizontal, hyperbolic arches, this time each subjected to an axial tension p. Again no vertical reactions result from this stress. The approximate distribution of the required horizontal reactions h have been indicated. As in the first loading, shears and forces normal to the arch axis must exist to complete the detailed equilibrium in the shell, and again these would modify the main stresses shown. If Σp is assumed equal to $H\,L/2 = P$, it is evident that the resultants from all reactions h of all arches are in equilibrium with the total edge load.

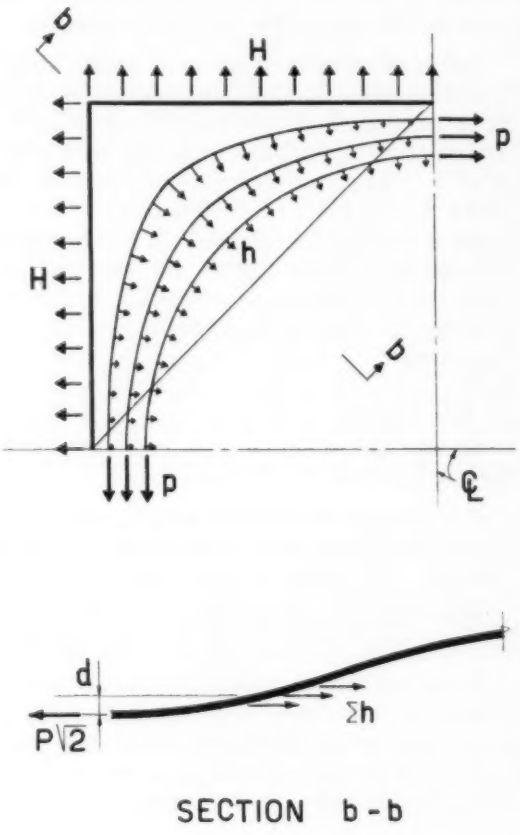

Fig. 15—Stresses due to horizontal edge loads. One-quarter of structural membrane unit is shown

By superimposing the two loading cases it becomes apparent that a greater part of the axial loads and their reactions will cancel out. It may be shown, however, that the shears from the two same loading cases will add together. Similarly, the heavy compression normal to the arch axis directly under the load $P\sqrt{2}$ will basically remain as in the first loading case.

Section a-a in Fig. 14 shows the resultant prestressing force $P\sqrt{2}$ and the horizontal reactions from the hyperbolic arches. Because these forces occur at various levels on the shell surface, radial moments are created. If the distance d between the location of the prestressing force and the resultant of the arch reactions were known, the total maximum radial moment could be determined as $M_r^{max} = P\sqrt{2}\ d$.

Section b-b in Fig. 15 shows the similar condition for the other loading case. A combination of the two loadings indicate that the greater part of the radial moments would cancel out. A crude estimate of the maximum radial moment due to the prestressing alone will be shown in the following example.

For a 20 x 20-ft dome unit with a height h = 2 ft a required prestressing force of 20,000 lb is assumed. If the maximum concrete stress is to be within an allowable limit of $f_c^{max} = 2000$ psi and assuming a rectangular stress block, the required concrete area to carry this load is 20/2 = 10 sq in. With a shell thickness assumed to be 2 in. this stress block would have to extend 5 in. in from the edge with its resultant located a minimum distance of 2½ in. away from the edge of the unit. It is apparent from Fig. 14 that the heaviest stress concentration occurs near the midpoint of each side of the unit. The slope of the shell normal to the edge at this location is $4h/L = 0.4$ and the distance d is therefore $d = 0.4 \times 2\frac{1}{2} = 1$ in. The maximum total radial moment is $M_r = 1 \times 20 \times 1.414 = 28.28$ in.-kips = 2.35 ft-kips. The distribution of this moment along the edge is not known. If uniform distribution over one-half of the total length is assumed, the radial moment $M_r =$ 0.24 ft-kips per ft results.

If a triangular stress block is assumed in place of the rectangular block and if the maximum stress at the edge is as before limited to 2000 psi, the radial bending moment would be 32 percent larger.

The local high compressive stresses at the corners of the units would have to be handled as in ordinary prestressed concrete design.

As stated earlier much theoretical and practical research is needed before the edge load effects will be fully understood. It appears, however, that some similarity exists between this problem and that of a deep beam design, and it is the author's opinion that they are not substantially more critical than that of the corresponding deep beam.

POSITIVE PROPERTIES OF THE STRUCTURAL MEMBRANE

Typical for all structural membranes is that the surface generally contains two or more sets of straight, horizontal lines. This is important architecturally, because it permits easy joining of a series of repeating shapes to cover wide areas. In the structural membrane described in Example 3 a uniform column spacing was used with a resulting square outline of these repeating shapes.

Like all thin shell structures, structural membranes depend upon horizontal thrust for their carrying capacities. Unlike most other thin shells, barrel shells for example, structural membranes do not need tie rods outside the surface itself. The straight lines described above provide an ideal location for tie rods, in form of pretensioned or post-tensioned strands. This simplifies anchorage, fireproofing, and similar problems in design and construction.

Another important feature of the structural membranes is the simplicity of the main curves in this shape. A template made up of two parabolas, one convex and the other concave, with each a height of $h/2$ and a span of $L/\sqrt{2}$ and connected with each other, if translated in the diagonal direction and sliding along the previously described straight lines, determines completely the Areas 1a, 2, and 3. These areas make up more than 80 percent of the entire surface. The only remaining part of the shell, Area 1, may be determined by rotating a second screed around the column, this template to conform in shape to the radial curve of the funnel. These two properties of the structural membranes are in the author's opinion the reason why this shape may become one of the most economical types known.

The simplicity in layout will be appreciated by any architect or engineer familiar with the problems in thin shell design. The same simplicity in the field does away with the costly layout work commonly connected with thin shell construction. The ease of forming or creating templates for forming is shown in construction recommendations.

CONSTRUCTION, ERECTION, DRAINAGE, AND WATERPROOFING OF STRUCTURAL MEMBRANES

Besides the standard method of cast-in-place construction, the structural membrane lends itself ideally to two modern construction methods. The first method is to cast the shell on the ground with the earth shaped to the proper form. After proper curing of the concrete the shell may then be stressed along the lines of the prestressing wires. Since all of its load is taken by column supports with no interferring tie rods at the support line it may now be lifted in place by a method similar to the lift slab method of erection.

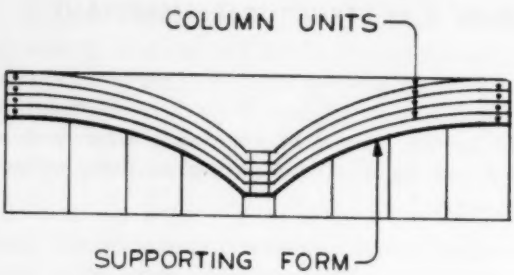

Fig. 16a—Precasting of units. Constant vertical thickness permits elements to be nested on jobsite casting beds

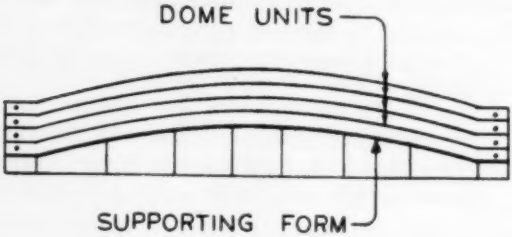

Fig. 16b—Casting arrangement for dome units

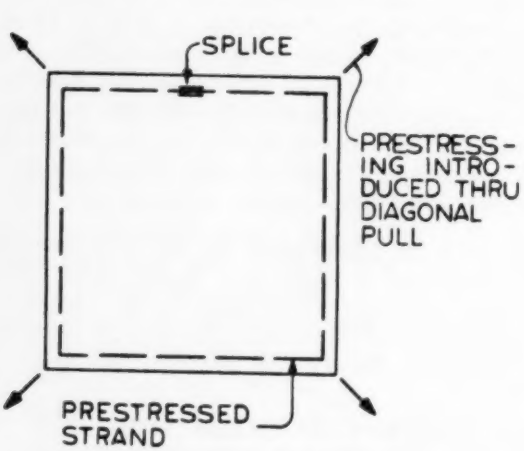

Fig. 17 — Arrangement of prestressing strand

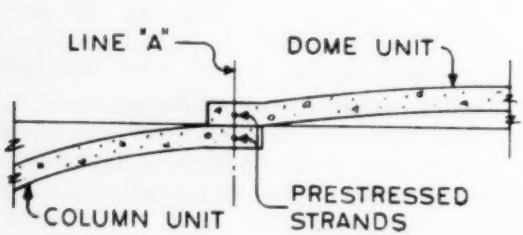

Fig. 18—Section through a construction joint

The second method of construction would primarily apply to structures with spans up to 40 or 50 ft with possibly 30 ft as the most economical. For this type the shells may be precast on a job site casting bed. The units will be of two types, referred to as the column units and the dome units. They will both have square outlines and the general dimension, L, of the units will be equal to $S/\sqrt{2}$, with S the column spacing in the structure. Due to the uniform stress typical for the structural membranes these units may be designed to have constant vertical thickness and therefore nested in the precasting bed (Fig. 16a and 16b).

The prestressing of each unit makes structural membrane units independent of the adjacent one for their load carrying capacity (Fig. 17). A series of simple construction joints are therefore possible. The typical one is shown in Fig. 18 with one unit merely resting on top of the edge of the other. Epoxy resins or other sealants placed in these joints would in most cases make them watertight. Mechanical clamping at the corners prevents the units from moving relative to each other. The column units are set on a flange or a seat on the column tops and cast-in-place concrete around the extension of the column provides a stiff moment connection between the column and the column unit.

The handling of the units during erection is simplified by the integral stability of each unit. The need for strong backs of vacuum pads is thereby eliminated. Placed in the building only the column unit requires light temporary shoring until the cast-in-place concrete has set up.

The over-all structural system is extremely stable for lateral loads as it consists of two intersecting series of three-hinged arches.

With carefully calked joints the roof should be watertight without any built-up roofing. The prestressing and the presence of only compressive stresses would eliminate the formation of cracks in the shell surface and consequently only a sprayed-on sealer would be required to prevent water from penetrating the shell.

Due to the shape, each column requires a down spout and the plumbing of the roof would normally cost more than in conventional design. Plastic tubing and fittings might offset some of this disadvantage and provide very efficient maintenance-free drainage.

CONCLUSION

The limitations of the use of a structural membrane are evident from its appearance. The surface must be curved. As with all thin shell work, this does not lend itself to multistory construction. A certain limitation as to selection of column spacing and arrangement exists. The problem of forming and finishing is more involved than the same for flat plates, ribbed or waffle slabs, and many other flat concrete floors or roofs.

At the same time the advantages are significant. Using very little concrete and steel, column spacings can be much greater than with most flat shapes. Regardless of spacing, the use of material is reduced. The beauty of the ceiling and architectural interest is increased. Combined window panels under the dome unit provide extremely interesting lighting and ventilation. Being under compression, little or no waterproofing should be required.

The structural membrane is almost ideally shaped from a structural standpoint. Its simplicity in design and construction makes it attractive even in an area where the labor cost is the major economical factor. With its pleasing lines it is felt that it might add beauty to an era geared principally for economy and production.

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American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by Apr. 1, 1961, for publication in Part 2 September 1961 JOURNAL.

CONCRETE BRIEFS

Notes from Field and Office

Concrete Test Molds and Concrete Capping Materials 851
Modulus of Elasticity of Concrete at Early Ages 854

Concrete Test Molds and Concrete Capping Materials

By NORMAN E. HENNING*

THE COMPRESSIVE STRENGTH of portland cement concrete is a measurable physical property when referenced to definite methods of test. Most of the procedures outlined in the ASTM standard methods, such as size of specimens, method of molding and curing, and rate of loading are sufficiently explicit and comparatively easy for the concrete technician to follow. Two factors which have a considerable influence on the compressive strength test results, the cylinder molds and the capping mixture, are still not sufficiently explicit and should be examined with a view to improve the uniformity.

Tests of paper molds for concrete cylinders were reported by Burmeister[1] and this work showed a reduction in strength of approximately 6 percent and 12 percent for two separate types of waxed paper molds when compared to concrete specimens made in steel molds.

The effect of type of capping material on the compressive strength of concrete cylinders was reported by Werner[2] and this paper concluded that cylinders capped with high-alumina cement gave higher crushing strengths for concretes of both high and low strengths than were obtained with other capping materials.

Although the waxed paper cylinder molds and the concrete capping materials used in this investigation comply with the current ASTM specification, it was advisable that tests be conducted to determine the reduction in strength as a matter of emphasis to certain local engineers and agencies. It was also advisable to evaluate the efficiency of the concrete capping material being predominantly used.

Scope of study

On each day for 5 consecutive days, concrete was batched in the laboratory of the same material and same mix proportion of sufficient quantity to cast 12 standard 6 x 12-in. concrete test cylinders. There was sufficient overage in the quantity of the batch to also determine the unit weights, slump,

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*Member American Concrete Institute, Vice-President, Twin City Testing and Engineering Laboratory, Inc., St. Paul, Minn.

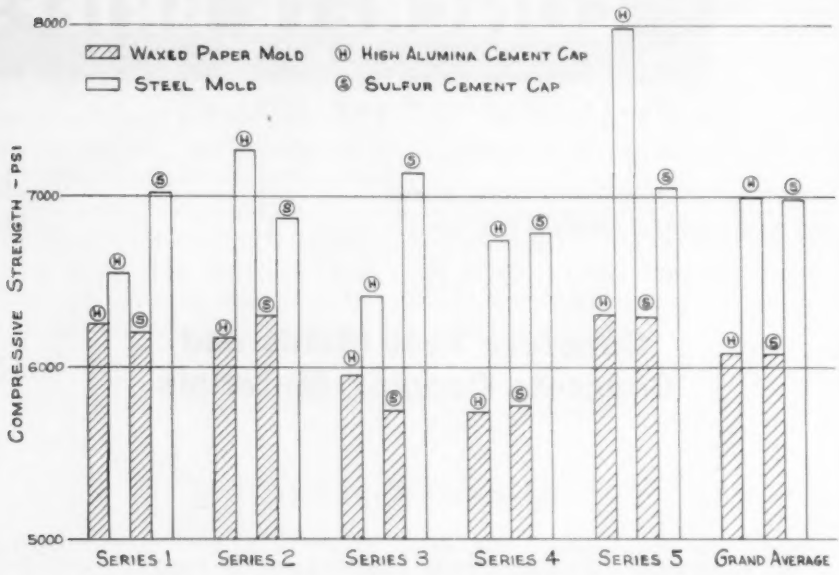

Fig. 1—Comparative average values of compressive strengths

and air content. The 12 specimens cast daily were handled as follows:

(1) Three specimens cast in waxed paper molds and capped with high-alumina cement

(2) Three specimens cast in waxed paper molds and capped with a sulfur-cement compound

(3) Three specimens cast in steel molds and capped with high-alumina cement

(4) Three specimens cast in steel molds and capped with a sulfur-cement compound

After 1 day in the molds, the specimens were removed and placed in the laboratory curing room until tested at an age of 28 days. These specimens were numbered consecutively, 1 through 60, and the same pattern of selection for the variables to which they were to be submitted was used.

Concrete mix proportions

The concrete mix proportion used was a mixture being used in a precast prestressed concrete plant to obtain a concrete having a compressive strength of 5000 psi at an age of 28 days. The properties of this mix were as follows: 705 lb Type I portland cement; 1185 lb No. 4 down concrete sand; 1930 lb ¾ in. to No. 4 gravel; and 0.25 lb Type 3a Pozzolith per sack of cement. Slump was 2 in. Net water-cement ratio was 4.2 gal. per sack.

Cylinder molds and molding

The waxed paper molds were a commercial product predominantly used in this area and have a sheet metal bottom. According to the manufacturer these molds comply with ASTM C 31-57. When filled with water for 24 hr these molds showed no leakage and had a gain in weight of 6.5 g, which would be an increase of 2.4 percent of the original weight of the mold. The total lineal expansion of the molds when filled with water for 24 hr was 0.040 in. The lineal expansion of these molds when filled with the concrete used in this study was 0.0205 in.

The steel molds in this study were the standard laboratory type comply-

TABLE 1 — TEST RESULTS

Type of mold	Waxed paper molds				Steel molds			
Type of capping compound	Specimen No.	High-alumina cement	Specimen No.	Sulfur-cement	Specimen No.	High-alumina cement	Specimen No.	Sulfur-cement
Series 1	1	6500 psi	2	6570 psi	7	6710 psi	8	7040 psi
	3	6290	4	6040	9	6305	10	6725
	5	5990	6	6005	11	6655	12	7245
	Avg	6260		6205		6555		7035
Series 2	13	6095	14	6535	19	7280	20	7245
	15	6395	16	6395	21	6775	22	6935
	17	6130	18	5970	23	7770	24	6410
	Avg	6175		6300		7275		6865
Series 3	25	6625	26	5885	31	6620	32	6865
	27	5760	28	5425	33	6235	34	7110
	29	5460	30	5920	35	6375	36	7420
	Avg	5950		5745		6410		7130
Series 4	37	5555	38	5215	43	6550	44	6970
	39	6105	40	5890	45	7385	46	6950
	41	5605	42	6215	47	6220	48	6375
	Avg	5735		5775		6735		6765
Series 5	49	5990	50	6500	55	7525	56	6220
	51	6750	52	5970	57	7510	58	6985
	53	6165	54	6395	59	8780	60	7910
	Avg	6300		6290		7970		7040
Grand average		6085		6065		6990		6965
Standard deviation		239		262		635		145
Coefficient of variation		3.5		3.9		9.1		2.1

ing with ASTM C 192-57. This steel mold has a machined, case hardened, metal base plate and the assembled mold and base plate was watertight.

The procedure for making and curing the compression test specimens was in accordance with ASTM C 192-57, with the exception, of course, that for half of the specimens cast a waxed paper mold was used.

Capping materials and methods

The sulfur-cement capping compound was composed of the following materials: 43.5 percent sulfur, 12.5 percent mineral filler limestone dust, 36.5 percent foundry sand, 6.75 percent fly ash, and 0.75 percent carbon black. This sulfur-cement compound is heated to 250 F $\pm$ 5 F and is not reused nor any attempt made at reclamation. The average compressive strength of this compound when tested on a 2 x 2 x 2-in. prism at an age of 4 hr was 8135 psi.

The high-alumina cement used for capping one-half of the specimens was Luminite cement. The 3-day compressive strength of this high-alumina cement when tested in 2 x 2 x 2-in. prisms was 9580 psi.

Both the sulfur-cement caps and the high-alumina cement caps were done in the usual laboratory manner with vertical capping jigs. Rather than using oil or a wax to prevent the caps from sticking to the capping base plate, a silicone release compound was used. (This compound is also used to prevent bonding of concrete to a plastic form liner.)

Test results

Testing was done in accordance with ASTM C 39-56T. Results are given in Table 1 and Fig. 1.

CONCLUSIONS

1. This study failed to reveal any difference in the compressive strengths obtained when capping was done with a high-alumina cement as compared to a sulfur-cement compound.

2. The reduction in strength with the use of the waxed paper molds was approximately 13 percent when ex-

pressed as a percent of the strength obtained in the steel molds.

3. The reduction in strength obtained with waxed paper molds should be more generally recognized, especially by design engineers. It can be seen that it is possible to follow ASTM procedures and still obtain results that are up to 13 percent low. On fully supervised projects where sufficient concrete technicians are available it is possible to use the steel molds. For much of the commercial testing this is not practical. The solution therefore must lie in either an improved disposable mold or a correction factor for a given brand of waxed paper mold as compared to a steel mold. For normal reinforced concrete strength requirements up to 4000 psi, the loss in strength can be compensated for by the use of a slightly higher cement factor. However, for concrete strengths in excess of 5000 psi, which are coming into increased usage, increasing the cement factor will not give the corresponding increase in strength and the additional factor of safety is therefore not obtainable.

4. The change in volume of voids and absorption of moisture by the waxed paper molds may be contributory causes to the reduction of strength. The change in volume of voids in a single cylinder would be about 18 cu cm. The tendency of the molds to expand may introduce capillary or tensile pressure in the water within the cylinder, as shown by Taylor[3] in undisturbed sampling methods. The absorption of water by the molds, though small, may cause drying as well as some capillary action on the surface of the cylinder. As the rate of hydration progresses, a certain amount of water will be made unavailable. The influence of these factors during the complicated setting process is not as yet exactly determinate, but certainly the changes may be appraised as significant.

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3. Taylor, Donald W., *Fundamentals of Soil Mechanics*, John Wiley & Sons, Inc., New York, 1956, p. 146.

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Modulus of Elasticity of Concrete at Early Ages

By T. W. THOMAS*

THE QUESTION FREQUENTLY arises in concrete construction: What effect will the early removal of forms have upon the deflection of the structure? Data are somewhat meager on moduli of elasticity at early stages. In this study, 6 x 12-in. cylinders were prepared for loading at ages of 2, 3, 7, and 28 days. Deformations were measured at the time of loading and the cumulative deformations were observed during an ensuing 7-day period under constant stress.

Concrete mix

The concrete was prepared from Type I cement using 3/4 in. maximum size gravel aggregate. The water-cement ratio was 7½ gal. per sack, the water content 325 lb per cu yd of concrete, and the cement content was 5.2 sacks per cu yd. The slump was 5½

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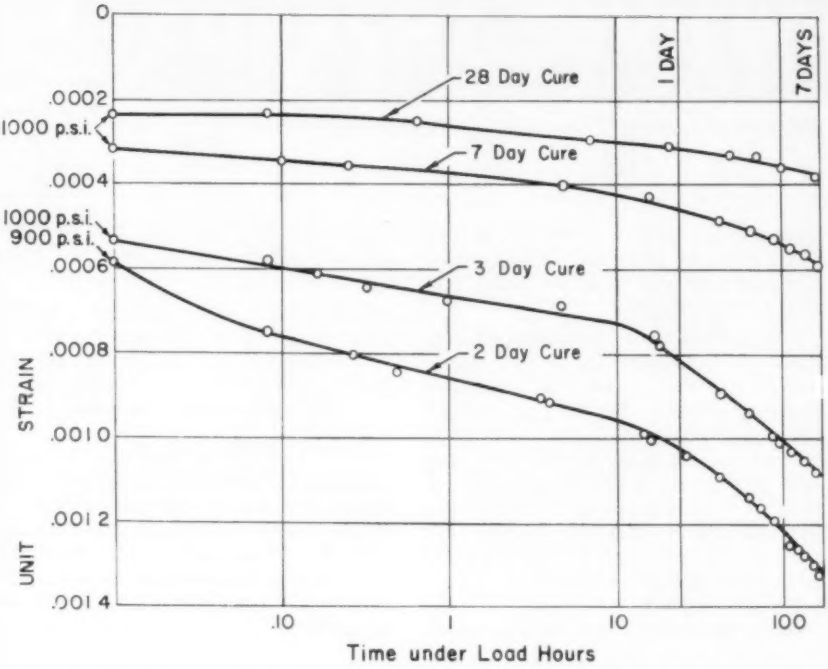

Fig. 1—Deformations under stress, after various periods of cure

in. The proportions by absolute volume are listed below:

$$w = 0.1928$$
$$c = 0.0920$$
$$a = 0.3212$$
$$b = 0.3940$$
$$1.0000$$

This concrete developed a 28-day strength of about 3600 psi.

PROCEDURE

It was desired to use the same unit stress in measuring deformations after each of the various periods of cure. A unit stress of 1000 psi was selected. This was used for curing periods of 3, 7, and 28 days. In the case of the 2-day curing period, however, it was necessary to reduce the stress to 900 psi. Curing was in moist air.

Deformations were measured with SR-4 strain gages, type A-9. In each case a dummy specimen of the same age and curing conditions was used, thus eliminating deformations due to shrinkage.

Fig. 1 shows the unit deformations for the various curing ages. These deformations start with the initial loading strain and then show the change during the 7-day period under load. At the time of loading, specimens cured for 2 days had 2½ times the deformation of those cured 28 days and 1.9 times those cured 7 days.

The effect of age at the time of loading is even more marked on deformations resulting during the 7-day period under load. The 7-day creep deformation for specimens cured 2 days was 5 times that for specimens cured for 28 days. This extreme difference is easily explained, however, when it is recalled that the 2-day cure specimen was stressed to 84 percent of its ultimate strength while the 28-day cure speci-

TABLE 1

Days moist cure	Ultimate strength, psi	Stress at which modulus was measured		Modulus of elasticity	
		psi	Percent of ultimate	At time of loading	Cumulative 7 days under stress
2	1068	900	84	1,550,000	684,000
3	1635	1000	61	1,890,000	925,000
7	2825	1000	35	3,230,000	1,700,000
28	3580	1000	28	4,280,000	2,650,000

men was stressed to only 28 percent of its ultimate strength. This information together with that for other ages is shown in Table 1.

Fig. 2 shows the instantaneous modulus of elasticity, the 7-day cumulative modulus, and the ultimate strength all as a function of time. It may be observed that the modulus at time of loading is somewhat more than 1000 times the ultimate strength. It should be emphasized that the cumulative modulus applies only to the dead load, or to that part of the live load present at those early ages.

Erzen* suggests a formula for the relation of modulus of elasticity at time t to that at age 28 days. For our data using Type I cement, constants 8 and 7 fit very well.

$$\frac{E_t}{E_{28}} = \frac{8}{7 + \frac{28}{t}}$$

E_t = modulus at t days

E_{28} = modulus at 28 days

Using the above formula and calculating modulus of elasticity at ages 2, 3, and 7 days from the 28-day value, we find the calculated values check the test values at those ages by +5.2 percent, +10.6 percent, and −3.7 percent, respectively.

*Erzen, Cevdet Z. "An Expression for Creep and Its Application to Prestressed Concrete," ACI JOURNAL, V. 28, No. 2, Aug. 1956 (*Proceedings* V. 53), pp. 205-213.

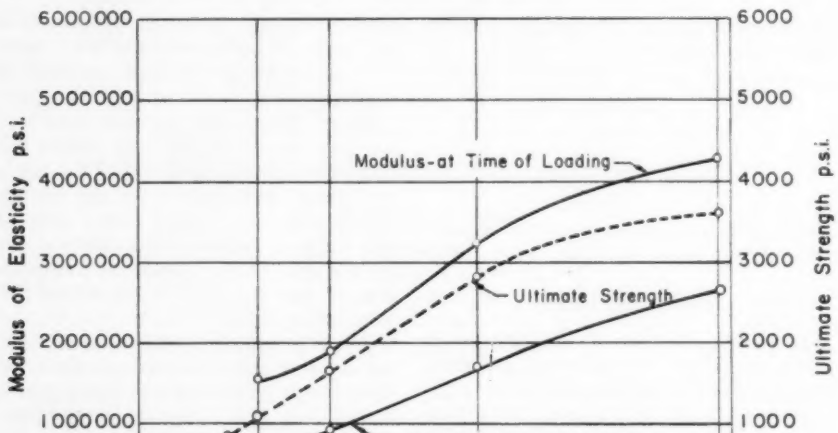

Fig. 2—Modulus of elasticity

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Modernising an 1875 bridge

Engineering (London), V. 189, No. 4892, Jan. 22, 1960, pp. 114-115

Reviewed by ARON L. MIRSKY

Brief description of an interesting job of reconstruction of an old two-track railroad bridge in England, particularly of the central pier. The existing pier was of masonry, pierced by an arch; to accommodate the new, deeper girders, the arch was temporarily braced internally by military trestling, the top was cut down, and a new prestressed concrete capping beam constructed. The abutments, which had been subjected to piecemeal strengthening over the years, were also strengthened further.

Construction

Building Esso refinery at Milford Haven

Engineering (London), V. 188, No. 4878, Oct. 16, 1959, pp. 346-347

Milford Haven refinery and jetty

The Engineer (London), V. 208, No. 5411, Oct. 9, 1959, pp. 388-389

Reviewed by ARON L. MIRSKY

Marine terminal 3500 ft long, catering to 100,000 dwt tankers, includes a jetty constructed of precast concrete members. Hollow prestressed piles up to 140 ft long were used, driven in some 80 ft of water (at high tide) despite strong tides. (Details of these piles and their manufacture are given in *The Engineer* and in *Engineering*, Oct. 9, 1959, p. 315.) There is a 16 ft wide roadway alongside the jetty, built of two 8 ft mesh-reinforced slabs spanning 35 ft between the pile caps; the joints between slabs were later filled with in-situ concrete reinforced with high-tensile steel. Reinforced concrete curbs serve to carry the live load and tie the structure together.

Subterranean Saint Pius X Basilica at Lourdes (La basilique souterraine Saint Pie X, a Lourdes)

Le Génie Civil (Paris), V. 136, No. 24, Dec. 15, 1959, pp. 501-506

Reviewed by ARON L. MIRSKY

Well-illustrated description of the layout, construction, and electrical-mechanical equipment. (For more detailed coverage, see paper by J. Chaudesaigues, *Travaux*, V. 42, No. 290, Dec. 1958, pp. 1003-1019, and V. 43, No. 291, Jan. 1959, pp. 9-22; "Current Reviews," ACI JOURNAL, July 1959, p. 71. For a brief description in English, see *Engineering News-Record*, V. 160, No. 21, May 22, 1958, pp. 28-29, 32.)

Broadcast House, Paris (La Maison de la Radio, a Paris)

Le Génie Civil (Paris), V. 137, No. 3, Feb. 1, 1960, pp. 53-61

Reviewed by ARON L. MIRSKY

Designed to centralize the radio and television facilities and operations of Radio diffusion-Television Francaise, this interesting structure is circular in plan (a hollow ring with a central core to which it is connected by a tower located on a radius). The substructure

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 7, Jan. 1961, *Proceedings* V. 57. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are *not* available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

is of reinforced concrete, with some of the chambers designed to serve as atomic shelters; the superstructure is variously of steel, reinforced concrete, and prestressed concrete.

Design and construction of the Queen Elizabeth Graving Dock at Falmouth

ARCHIBALD E. UNDERWOOD, JOHN D. NORFOLK, and JOHN N. EATHORNE, *Proceedings*, Institution of Civil Engineers (London), V. 15, session 1959-60, Jan. 1960, pp. 49-64
Reviewed by ARON L. MIRSKY

Designed in 1954 to meet the needs of an expanding oil industry, dock was redesigned and enlarged in 1957, when construction was well along, to accommodate vessels then being ordered. Dock thus has 850 ft length, 136 ft width between sidewalls, 130 ft wide entrance, and 36 ft depth over sill at high water; vessels of 85,000 tons dead weight can be handled.

Construction Technique

Economies with composite construction

F. A. LEFEVER, *Engineering* (London), V. 189, No. 4911, June 3, 1960, p. 752
Reviewed by ARON L. MIRSKY

Brief presentation of the basic principles and the advantages of composite construction. Load factor analysis (steel fully plastic, concrete fully developed in compression) is advocated as more realistic than elastic design. Prestressing plus use of precast slab units are suggested as the most promising solution to the problem of composite action in regions of negative moment.

Some post-war works of the Thames Conservancy

HERBERT C. BOWEN and WILLIAM E. FOSTER, *Proceedings*, Institution of Civil Engineers (London), V. 14 (Session 1959-60), Nov. 1959, pp. 255-274
Reviewed by ARON L. MIRSKY

Describes three interesting reconstruction jobs on the Thames: the Chertsey Weir (built about 1812), half of which collapsed during the operations, and the locks at Old Windsor and at Cookham (originally constructed in 1838 and 1830, respectively). Prestressed precast concrete sheet piles were used in the reconstructed Old Windsor lock.

Granolithic floors: A new durable mix

F. J. COSFORD, *Engineering* (London), V. 189, No. 4903, Apr. 8, 1960, p. 476
Reviewed by ARON L. MIRSKY

To obviate maintenance problems, new approaches include (1) use of only enough water, in both concrete base and granolithic topping, to hydrate the cement; (2) complete compaction of the base and topping to obtain maximum density; and (3) vertical joints between floor and walls, foundations, and similar elements.

Dams

Dutch delta plan

The Engineer (London), V. 208, No. 5410, Oct. 2, 1959, pp. 337-352
Reviewed by ARON L. MIRSKY

A series of four articles, covering the plan for protecting the islands of southwestern Holland from storm surges and ground water salinity, and three of the major structures (Haringvliet Dam, Veersche Gat Dam, and the surge barrier), respectively. Completion date of the project is 1978, at an estimated cost of some $560 million.

Calculations of hollow dams and buttressed dams (Le calcul des barrages evides et a contreforts)

CLAUDE BOURDON, *Mémoires*, Société des Ingénieurs Civils de France (Paris), V. 112, No. 5, Sept.-Dec. 1959, pp. 353-358
Reviewed by RICHARD CEBULA

Starting from the general equations of equilibrium for a horizontal section taken between any point of a symmetrical pier and the downstream face of a hollow dam or buttressed dam, the author shows that the products of each of the elementary stresses (normal, tangential, and horizontal) by the width of the pier may be expressed in terms of a second-order derivative of the moment, at the point considered, of the resultant of the elastic

effects acting between this point and the face. This "moment" function plays in the plane elastic problem a role similar to the Airy function, with which it becomes identical if the mass effects are ignored. From it the author derives a general method for calculating both the elementary stresses and the principal stresses at any point of a hollow or buttressed dam. The article ends with brief suggestions for checking the safety conditions and calculating the foundation pressure.

Geological features and foundation treatment at the Beechwood Development

I. D. MacKenzie and E. L. Brown, *The Engineering Journal* (Montreal), V. 42, No. 12, Dec. 1959, pp. 54-62
Reviewed by Aron L. Mirsky

Describes geology, foundation conditions, and curtain and blanket grouting on the Beechwood Development, a hydropower plant on the St. John River. Structures consist of gravity type concrete sluice and regulating sections, a fishway, an intake and powerhouse, and a gravity type abutment (western end) with a compacted earth embankment at the eastern end. Grouted condition of cores obtained during drilling of secondary holes in the fault zone, and observed inflow in drainage system and observation wells more than a year after head was raised, indicate effectiveness of grouting program.

Major hydroelectric plant at Bratsk in Siberia (Das sibirische Grosswasserkraftwerk Bratsk)

Helmut Leutelt, *VDI Zeitschrift* (Düsseldorf), V. 101, No. 19, July 1, 1959, pp. 788-792
Reviewed by Aron L. Mirsky

Hydroelectric scheme comprising five dams on the Angara (the outlet of Lake Baikal) and two on the Jenissei (one of them after its confluence with the Angara) is designed to produce large amounts of power for use in exploiting the mineral resources of the area and for feeding into a projected power grid. The power plant near Bratsk, now under construction, is slated to produce 3.6 million kw* at an average annual rate of 21.7 billion kw-hr; the first turbine is scheduled to go on the line around 1961 and the plant is to be in full operation by 1963 (1962 according to some sources).

Especially interesting as a West German appraisal of some Russian claims.

**Engineering News-Record*, Sept. 17, 1959, p. 37 gives it a rating of 4.5 million kw.

Design

Shapes in shells

The Engineer (London), V. 208, No. 5416, Nov. 13, 1959, p. 618
Reviewed by Aron L. Mirsky

A page of pictures, with brief text descriptions, of three interesting shell designs by Rene Sarger: covered markets at Royan and Nanterre, and a church at Royan.

Experimental study of beams on elastic foundations

Robert L. Thoms, *Proceedings*, ASCE, V. 86, EM 3, June 1960, pp. 107-118
Author's Summary

The design and use of a structural model for the solution of shear and moment in a beam on an elastic foundation is presented. Through use of the model, a typical problem of a laterally loaded pile foundation is solved as an example.

Underground structures subject to to air overpressure

Ernest T. Selig, Keith E. McKee, and Eben Vey, *Proceedings*, ASCE, V. 86, EM 4, Aug. 1960, pp. 87-103
Authors' Summary

An analytical procedure is presented for determining the damage to underground structures induced by pseudo-steady state air overpressure. By introducing appropriate assumptions from the fields of soil mechanics and structural dynamics, it is possible to derive analytical expressions that relate the parameters of the structure, the soil, and the loading as they affect the failure of the structure. The assumptions are stated and the derivation of the analytical procedure is presented. A separate section is devoted to the ap-

plication of the procedure for the cases of analysis of the vulnerability of an underground structure and the design of an underground structure for any desired level of protection.

Stresses and deformations in elastic frames

ALFRED J. S. PIPPARD and LETITIA CHITTY, *Proceedings*, Institution of Civil Engineers (London), V. 16 (Session 1959-60), June 1960, pp. 171-188

Reviewed by ARON L. MIRSKY

Authors develop simple analytical method, applicable to trussed structures (plane or space; simply-stiff or redundant), for the determination of the forces in all members and of the component displacements of all joints. Method involves forming two sets of equations, the first being the tension-coefficient equations and the second involving the joint displacements; solution is by successive substitution. Method is especially useful for obtaining influence lines for hyperstatic structures. Four examples are given, of types which were once exclusively of steel, but which are now being designed — and constructed — in prestressed concrete.

Practical method of designing thin shell structures of double curvature (Methode pratique de calcul des voiles minces a double curvature)

P. CSONKA, *La Technique Modern-Construction* (Paris), V. 15, No. 8, Aug. 1960, pp. 311-313

Reviewed by ALEXANDER M. TURITZIN

The author's purpose is to simplify the analysis of the thin shell roofs supported by arches along their edges. He believes that this type of roof has a pleasing effect, but it is rarely used because of the complexity of its analysis.

In his previous publications, the author has already presented his method of indefinite form for the approximate solution of thin shells. In the function which characterizes the shape of the middle section of the shell, this method brings in a free parameter in addition to the data previously obtained. Definite values are given to the free parameter, thus making it possible to make relatively simple computations with sufficient accuracy.

In the second part of the article, the author applies his approximate method to the analysis of a shell of double curvature with a rectangular projection on its base. In conclusion a bibliography is given to previous publications of the author on the same subject.

Response of an elastic structure to blast-type loading

E. T. SELIG, *Journal*, Boston Society of Civil Engineers, V. 47, No. 2, Apr. 1960, pp. 169-182

Reviewed by ARON L. MIRSKY

Author develops a method for rapid investigation of the effects of structural and blast-loading parameters on the maximum displacements, stresses, and strains. Numerical values of these parameters must be determined by the designer to fit the problem at hand.

Method is applicable to structures which may be represented by systems with one degree of freedom, and to blast-type loads which may be idealized to an instantaneous impulse plus a suddenly applied, linearly-decreasing force.

A chart of the maximum dynamic displacement factor is given, to aid in investigating the effect of varying the parameters without having to repeat the lengthy solution of the equation of motion each time.

Pavements

Very fine cements and blast furnace cements in cement concrete highway pavements (Ciments Ultra Fins et Ciments de Haut Fourneau dans les Revetments Routiers en Beton de Ciment)

LEON BLONDIAU, *Revue des Matériaux* (Paris), No. 529, Oct. 1959, pp. 231-246; No. 530, Nov. 1959, pp. 267-274

Reviewed by PHILLIP L. MELVILLE

An experimental road was built in 1954 in Belgium with the following cements: standard blast furnace, high strength blast furnace, high strength portland, high sulfate, quick setting blast furnace and quick setting port-

land. The quality and quantity of aggregates were kept constant but less cement was used for the fast setting finer cements. Both continuous and discontinuous gradations were used. Testing was on beams in flexure and cubes cut from beam ends in compression as well as from specimens cut out of the as-built pavement. It was concluded that the important criteria for highway pavements are modulus of rupture and low shrinkage. The concrete with high sulfate cement had highest strength with somewhat less shrinkage. Blast furnace cements gave identical performance to that of high strength portland. High fineness cements have the advantage to allow a 20 percent reduction by weight of cement for equal performance.

London-Birmingham Motorway

Proceedings, Institution of Civil Engineers (London), V. 15 (Session 1959-60), Apr. 1960, pp. 317-410

Reviewed by ARON L. MIRSKY

General motorway plan

JOHN F. A. BAKER, pp. 317-332

Economic and traffic studies

WILLIAM GLANVILLE, pp. 333-352

Luton-Dunchurch: Design and execution

OWEN WILLIAMS and OWEN T. WILLIAMS, pp. 353-386

Luton-Dunchurch: Construction

WILLIAM K. LAING, BEAUMONT H. BROADBENT, and JOHN M. FISHER, pp. 387-400

St. Albans By-pass, Part I: Design

CHARLES H. FROLLIOTT, pp. 402-406

St. Albans By-pass, Part II: Construction

THOMAS MCMILLAN, pp. 406-410

Five papers covering various aspects of Britain's first high-speed, limited-access, restricted-use motorway (superhighway). The genesis and growth of the idea, the detailed investigations justifying the plan, the design and construction of the highway and its structures, and suggestions for improving future projects, are discussed, by representatives of the public officials, consulting engineers, and contractors involved. The scheme and results are impressive; reviewer, undoubtedly as a result of conditioning, admits to being particularly struck by the appearance of the various bridges (reportedly passed on by the Royal Fine Arts Commission) and by the large-scale use of mass concrete in their construction. See also paper by W. Kirby Laing in *Engineering*, Sept. 11, 1959, pp. 175-179; "Current Reviews," ACI JOURNAL, V. 32, No. 3, Sept. 1959 (*Proceedings*, V. 56), p. 345.

Polyethylene sheets as curing covers for concrete pavements

Research Report RR 60-1, Engineering Research Series, Bureau of Physical Research, New York State Department of Public Works, Albany, Mar. 1960

HIGHWAY RESEARCH ABSTRACTS V. 30, No. 9, Oct. 1960

In this study, white opaque, 4-mil polyethylene sheeting and clear polyethylene sheeting of 4 mil and 1¼ mil thickness were used for curing concrete pavements. All sheeting performed satisfactorily in the following respects:

1. It kept the surface of the slab uniformly moist throughout the curing period.

2. It was obtained in lengths varying from 100 to 500 ft and in single widths between 12 ft and 24 ft 3 in.

3. Its light weight and imperviousness to water made it easy to handle and reuse.

4. It did not detract from the compressive strength as evidenced by the fact that cores taken from concrete cured with polyethylene had a slightly greater average compressive strength than cores taken from concrete cured with waterproof paper.

The average daily temperature range at the pavement surface under opaque, 4-mil polyethylene sheeting was slightly greater than the temperature range beneath waterproof paper. This increase is not considered detrimental. However, the temperature range beneath clear polyethylene was considerably greater than that under paper and although no cracking of the concrete due to shrinkage was observed

these extreme temperature fluctuations are to be avoided.

Based on this experiment which gives consideration to the compressive strength of cores, temperature fluctuations, and general condition of the pavement surface, it appears that the pavement was adequately cured by white opaque, 4-mil polyethylene.

Precast Concrete

NYCTA: New platforms lower station upkeep

Railway Age, V. 148, No. 23, June 6, 1960, p. 14
Reviewed by ARON L. MIRSKY

New York City Transit Authority has embarked on a 10-year program of replacing wooden platforms with precast concrete slabs on its elevated stations. Although first cost is higher, savings over the estimated 50-year life span of the slabs will be appreciable. Additionally, the slabs are much safer, particularly for devotees of high-heeled shoes.

Prestressed Concrete

Two-span haunched continuous beam in prestressed concrete

REGINALD G. ROBERTSON, *Proceedings*, Institution of Civil Engineers (London), V. 15 (Session 1959-60), Mar. 1960, pp. 255-272
Reviewed by ARON L. MIRSKY

For economy, the maximum allowable fiber stresses must be attained at as many sections as possible. For the two-span continuous beam with haunches, this means four critical sections: (1) in the central portions of the spans, due to range in moment (stress reversal), (2) at the intermediate support, (3) at the critical section for maximum moment in the span, and (4) at the end of the haunch.

Such equalization of stresses can be accomplished only by prestraining—that is, by superimposing initial secondary moments on the elastic moments due to the given loading. Method previously derived by author (*Proc.* ICE, V. 10, May. 1958, pp. 39-58; "Current Reviews," ACI JOURNAL, V. 32, No. 1, July 1959 (*Proceedings*, V. 56), p. 90) is here generalized to include this secondary moment and expressions are derived for haunch dimensions and cable profile and concordance. A design chart is given for a beam with an I-shaped cross section with a thin web, and its use illustrated by a worked example.

Author concludes a two-span continuous beam is more economical than two simple spans for longer spans, a haunch (length increasing with length of span) is necessary for economy, and this haunch is larger than required for the two-span articulated beam, with the latter being slightly more economical only for very long spans.

Experienced crew takes prestressed girder job in stride

H. K. GLIDDEN, *Roads and Streets*, V. 103, No. 6, June 1960, pp. 74-78, 83

Discusses the construction and placement of 248 large post-tensioned concrete girders for a highway bridge on a segment of the Interstate 70, east of Denver. Topics reviewed are: girder design, organization and scheduling, casting, stressing, grouting, and placement of girders, and test results.

Field testing and analysis of two prestressed concrete girders

ADRIAN PAUW and JOHN E. BREEN, *University of Missouri Bulletin* (Engineering Experiment Station Series No. 46), V. 60, No. 52, 1959, 142 pp.

A research program to study the field behavior of a 99-ft clear span prestressed concrete pedestrian overpass was undertaken; objectives of the research were the collection and analysis of field strain and deflection measurements for the prestressed, post-tensioned girders and cast-in-situ walkway slab, and the comparison of the actual behavior of the structure with that predicted from the design assumptions.

This report describes the test program developed, including the control tests used in determining the design constants for calculating theoretical deflections and strains. The field ob-

servations are presented in tabular and graphical form and the correlations between observed and computed values are discussed.

Hook Road underpass

The Engineer (London), V. 209, No. 5430, Feb. 19, 1960, p. 301

Reviewed by Aron L. Mirsky

Underpass, second of its kind in the London area, consists of twin prestressed concrete bridges over the Kingston bypass (A. 243) in Surrey, set in an elliptical traffic "circle" provided with four slip ramps. Layout and construction were severely restricted by site and traffic conditions. Asphalt wearing surface is electrically heated to prevent ice formation on slopes of the cut; control is automatic.

Stresses and factors of safety in prestressed concrete

P. W. Abeles, *Concrete and Constructional Engineering* (London), V. 55, No. 5, May 1960, pp. 191-193

Factors of safety recommended in British Standard Code No. 115 (1959), "The Structural Use of Prestressed Concrete in Buildings," are compared with corresponding recommendations in the United States and Germany. Continuation of an article in the February issue dealing with working stresses; see "Current Reviews," ACI Journal, V. 32, No. 5, Nov. 1960 (*Proceedings* V. 57), p. 602.

On the prestressing of the slab-type load-carring elements of massive structures (Zur Vorspannung der Flaechentragwerke des Massivbaus)

Georg Knittel, *VDI Zeitschrift* (Düsseldorf), V. 102, No. 7, Mar. 1, 1960, pp. 253-259

Reviewed by Aron L. Mirsky

Technical albeit nonmathematical exposition of shells (one extreme) and flat plates (the other extreme of slab-type structural elements), and the application of prestressing to reduce or eliminate the effects of secondary (boundary) stresses. Structural types discussed are: rotationally-symmetrical cupolas (domes), cylindrical shells, supported in various ways, containers (tanks), suspended roofs, cantilevered plates, and pavements.

Structural Research

Photographic method for determining the deflected shapes of buckled plates

Chia-Yao Yuan and Desi D. Vasarhelyi, *The Trend in Engineering at the University of Washington*, V. 12, No. 1, Jan. 1960, pp. 12-15

Reviewed by Aron L. Mirsky

An interesting variation of the grid method of experimental stress analysis: before-and-after-loading photographs of the shadows cast by a grid (network) of strings under a single strong light on the member under study may be analyzed by photogrammetric methods, or may be utilized directly to give a visual indication of the deformed surface and its mode of action. Method was developed in conjunction with study of buckling of webs of plate girders; due to the simplicity of the apparatus and the ease of obtaining results, it should be useful in studies of actual structures (or large portions thereof).

Newtonian theory of axial elasticity

A. N. Procter, *Engineering* (London): Part I, V. 187, No. 4867, June 19, 1959, pp. 798-800; Part II, V. 188, No. 4868, Aug. 7, 1959, pp. 3-4; Part III, V. 188, No. 4869, Aug. 14, 1959, pp. 37-38. Discussion: Sept. 18, 1959, pp. 195-196, and Oct. 23, 1959, pp. 367-368

Reviewed by Aron L. Mirsky

An interesting and thought-provoking paper. Author presents evidence for existence of transverse *stresses* in uniaxially-loaded members fulfilling all the usual assumptions. Major evidence is the form of failure of concrete specimens in compression, with role of height-diameter ratio in determining whether failure is primarily in compression (crushing) or shear investigated by energy methods; photoelastic and ultrasonic evidence is also presented. Paper will undoubtedly serve to answer a good many questions —and raise still others—in thoughtful engineers' minds.

For some additional comments on the splitting failure of concrete cylinders, etc., the interested reader is referred

to "Some Aspects of the Strength of Concrete," by A. M. Neville, *Civil Engineering and Public Works Review* (London), V. 54, No. 639-641, 1959: Oct., pp. 1153-1156; Nov., pp. 1308-1310; Dec., pp. 1435-1439, and to Jaeger's paper reviewed below.

Rock failures at low confining pressures

J. C. JAEGER, *Engineering* (London), V. 189, No. 4897, Feb. 26, 1960, pp. 283-284
Reviewed by ARON L. MIRSKY

Triaxial tests at low confining pressures (0 to 3000 psi) show that the basic mechanism of failure of several rocks (and 2:1 cement mortar) is slip on a single shear plane for low as well as for high pressures, although other forms of failure (e.g., slabbing with tensile cracks) may also occur, depending on the support conditions at the ends of the specimens.

General

On the formation of dust in clinker cooler (Sur la Formation de Poussieres dans les Refroidisseurs a Clinker)

R. ALEGRE and P. TERRIER, *Revue des Matériaux* (Paris), No. 536, May 1960, pp. 109-112
Reviewed by PHILLIP L. MELVILLE

Chemical analysis of dust samples from clinker coolers indicates that the particles are different from those coming from the stacks and also from the clinker itself. Study under the microscope shows that C_3S crystals may have split due to thermal stresses in the dust.

Actual examples of sea-going ship construction in reinforced and prestressed concrete (Exemples de realisations de constructions navales de mer en beton arme et precontraint)

HENRY LOSSIER, *Le Génie Civil* (Paris), V. 137, No. 8, Apr. 15, 1960, pp. 170-173
Reviewed by ARON L. MIRSKY

Reinforced concrete vessels, the first practical example of which is ascribed to Francais Lambot (1850), became prominent in the First World War. Article describes briefly author's seagoing barge of reinforced concrete (1918), and then covers the prestressed concrete outfitting dock "Elbe 17" at Hamburg, which was constructed upside down in the initial stages, righted, and completed. Author's conclusion: if the dead weight of concrete construction does not militate against navigable vessels, why not utilize such construction for naval facilities which remain moored, especially in prestressed concrete, which results in reduced weight?

Summary of engineering research—1959

University of Illinois Bulletin, V. 57, No. 43, Feb. 1960, 146 pp.

Summarizes research in the College of Engineering at the University of Illinois. Each listing of a research project gives the project title, names of the investigators, a brief general statement of the work being done, and a list of publications and those that have resulted from the research during the year covered by this summary.

Can the construction contract cover detail engineering costs?

WARNER HOWE, *Consulting Engineer*, V. 15, No. 1, July 1960, pp. 86-88
Reviewed by ARON L. MIRSKY

By its nature, concrete and reinforced concrete construction demand different treatment from, say, steel construction. To protect himself, his client, and the public, the consulting engineer on concrete structures should prepare the shop and construction drawings as well as the design (contract) plans, and supervise construction; to get paid, he may do as Memphis engineers do, have the contractor's bid include an allowance covering this, to be paid by the contractor to him. Author sees no real conflict with codes of ethics; the editor of *Consulting Engineer* is somewhat less sanguine (p. 34).

ACI NEWS LETTER

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memo from the President:

"A FAIR EXCHANGE"

What is ACI? What does it do for you, and what can you do for it?

Primarily ACI is its members—ten thousand people working together to get more done than ten thousand people could do separately. Our common purpose is, to quote from the charter:

"To organize the efforts of its members for a nonprofit public service in gathering, correlating, and disseminating information for the improvement of the design, construction, manufacture, use, and maintenance of concrete products and structures."

Some of the ways in which ACI carries out that purpose are by encouraging research and creativity, developing recommended practices and codes, publishing useful information, sponsoring meetings, and generally improving communication regarding concrete. Through committees, publications, meetings, and awards it gives stimulus to people with ideas and gives weight to desirable practices. It provides a concerted attack on technical problems. It supports better and more efficient construction. It improves public understanding and respect for concrete.

What are the products of ACI's many activities? Through the technical committees ACI promotes research, fact-finding, and writing which lead to recommended practices, codes, specifications, reports, and various other publications. The main stream of our publications program is the monthly ACI JOURNAL, which automatically goes to all members and which constitutes also the annual *Proceedings*. Widely

used special publications include the *Reinforced Concrete Design Handbook*, the *Detailing Manual* (ACI 315-57), the *ACI Manual of Inspection*, and the indexes of publications, among others.

Another product of ACI activities is the cross-communication and cross-fertilization that occur at the two annual meetings. The days are crowded with technical committee meetings (open to the membership), technical sessions, Institute business, and exhibits. And all the time shop talk is going on, in the corridors, the hotel rooms, and the restaurants. Many members say that their greatest benefit from convention attendance is the personal contacts and individual discussions.

To provide still greater opportunity for meeting together, the ACI members in four areas have formed local chapters; and several other chapters are in process of formation. Chapters have programs and projects of particular interest in their localities, and they enjoy the benefits of personal contacts and shop talk with many with whom they deal directly.

What do individual members get out of ACI, and what do they do for ACI? There's a fair exchange, an interactive process. Of course, everybody reads parts of the JOURNAL, according to their special interests; and many also draw directly or indirectly from ACI special publications. Many members participate in committee work, write papers, and attend meetings. All members have the satisfaction of supporting the Institute activities with their dues. Concrete can do many wonderful things, but it admittedly has some problems. Getting together to work out these problems is a challenge which undoubtedly adds many to our membership.

Joel W. Kelly
President

Nordby to head newly organized Committee 403

The Institute Board of Direction has approved the Technical Activities Committee recommendation for the formation of Committee 403, Adhesives for Concrete, with Gene M. Nordby, professor and head, Department of Civil Engineering, University of Arizona, as chairman.

The committee has been assigned the following mission: The committee is to study, assemble, and report information on the suitable *use* of adhesives for structurally joining concrete elements, restoring deteriorated concrete, or providing a wearing surface or similar uses. These adhesives include epoxy resins, liquid polyesters, and chemical concretes including new materials as they are developed. Particular attention is to be placed on reporting safe handling practice and proper preparation of these often toxic materials. Promoting and encouraging research on the proper use of these materials in conjunction with concrete will be emphasized.

Dr. Nordby is a member of ACI Committee 115, Research, and Committee 215, Fatigue of Concrete. He was chairman of Committee 215 from 1957-1959. Committee members have not yet been appointed to the newly organized Committee 403.

Lynn joins Columbia Cement sales staff

David E. Lynn has joined the Columbia Cement Corp., Pittsburgh, as a sales representative in the northern Ohio territory. He will be associated with the firm's Cleveland district sales office in his new position.

Highlights from ACI Regional Meeting—Tucson

AN OUTSTANDING TECHNICAL PROGRAM drew more than 350 registrants to the ACI 13th Regional Meetings, Pioneer Hotel, Tucson, Ariz., Oct. 31 to Nov. 2, 1960. Sessions were devoted to construction, research related to design, and design. Two sessions were sponsored jointly by ACI and the American Society for Testing Materials on research and research applications in the concrete field.

TECHNICAL PROGRAM

The papers presented at the regional meeting are summarized briefly in this News Letter. It is anticipated that many of these will be submitted for possible publication in the ACI JOURNAL later this year.

One day was devoted to joint sessions with ASTM. The construction session was chairmanned by Frank Stubbs, professor of civil engineering, University of California, Berkeley. D. R. Butler, Integrated Incorporated, general and building contractor, Los Angeles, and president of the ACI Southern California Chapter, acted as chairman of the session on research in design. The ACI design session was chairmanned by B. J. Shell, San Xavier Construction Co., Tucson. The ASTM research session was under the chairmanship of John T. Young, materials consultant, Arizona Aggregate Association, Phoenix. Cedric Willson, vice-president of engineering, Texas Industries Inc., Dallas, was chairman of the ACI-ASTM research session.

Missile base construction problems

Lt. Col. G. H. Andrews, Directorate of Civil Engineering, Headquarters, Strategic Air Command, Offutt AFB, Omaha, Neb.

A discussion of the configurations of the Titan missile system and the various major design problems encountered. Design and construction had to take into account considerations of blast resistant construction; operation under attack; forces due to missile firing; the critical requirements of servicing and firing missiles; and modifying missile sites to keep pace with the rapid developments being made in types of missiles, fuels, electronic guidance systems, and other related equipment and facilities. Construction and design management was discussed and typical construction operations were illustrated at various phases of completion of the missile bases.

Timber falsework design and usage

George R. Birkemeier, chief engineer, Timber Structures, Inc., Portland, Ore.

Discussed timber falsework as an economical method for supporting forms in large construction projects such as hangars, buildings using thin-shell concrete roofs, bridges, overpass-

A pause in discussion for (l. to r.) J. Bruse Vesey, Honolulu, Hawaii; ACI President Joe W. Kelly; and ACI Secretary-Treasurer William A. Maples

Tucson regional meeting committee (l. to r.) — Edward Mangotich, Portland Cement Association (exhibits); Gene M. Nordby, University of Arizona (general chairman); Russell Taylor of Taylor-McFarland-Johnson, Consultants (luncheon and banquet); Andrew Marum, Marum and Marum, Consultants (membership); Howard Harrenstien, University of Arizona (educational and student contest); Leland Lawrence, Friedman and Jobusch, Consultants (tickets); and Andrew Ross, University of Arizona (vice-chairman and technical program). Not in the photo: Gene Anderson, civil engineer (sessions); Kenneth K. Kienow, University of Arizona (registration); Maurice LaGaard, University of Arizona (field trips); and ladies' events co-chairmen Mesdames, Nordby and LaGaard

es, tunnels, culverts, and other structures using cast-in-place concrete.

Also discussed was the use of timber falsework as centering for precast concrete units that must be assembled in place; the various types of material, surface loads, decentering procedures, and the use of traveling falsework.

Effects of earthquakes on reinforced concrete structures

Roger Diaz de Cossio and Emilio Rosenblueth, respectively, research associate professor, and research professor and director, Institute of Engineering, University of Mexico, Mexico, D.F., Mexico

Descriptions and illustrations were given of failures of reinforced concrete members due to earthquakes in Mexico City (July 1957) and Coatzacoalcos (August 1959). By far the most common mode of failure was diagonal tension in girders as well as columns. Very seldom was a clear-cut flexural failure observed. In many cases, the failures could be correlated with poor construction techniques, with lack of a seismic design, or with oversimplifications in design. However, in some cases the failures pointed to weaknesses of current building codes.

The world's largest lift slab: Motorola Building, Phoenix

Charles R. Magadini, consulting structural engineer, Phoenix, Ariz.

The reasons for choosing this type of construction were given and design methods were described. Revisions for slab movement due to shrinkage, creep, and plastic flow were discussed. Field problems encountered during construction were mentioned and the methods used to solve these problems were described. The post-tensioning devices and their method of use and handling, were illustrated, as were the lifting methods and method of attachment of the slabs to columns of both temporary and permanent nature.

Solution of difficult concrete structural problems through model analysis

Jack R. Janney, Engineers Collaborative, Chicago, Ill.

Interest in thin shell, folded plate, and other forms of concrete construction which result in efficient use of

Expounding on his favorite subject, ACI President Joe W. Kelly (far left) talks "concrete" with (l. to r.) Roger Diaz de Cossio, Instituto de Ingenieria, University of Mexico, Mexico City, Mexico; Arturo Delgado, University of Sonora, Hermosillo, Mexico; and S. R. Bhonsle, Bombay, India

materials has been manifested by the recent appearance of a number of structures which are unique in appearance and structural function. Such structures often present difficult problems of design and analysis. Consequently, the configuration of the structure is often confined to forms for which mathematical solutions exist or may be readily resolved.

The use of electronic computers has increased the practicality of design and analysis for many of these forms. Recent experience has shown that the next step beyond computers is making use of structural models. Design and analysis outside the realm of present knowledge of elastic theory often may be accomplished with one or more properly designed and executed model analyses. Furthermore, the use of structural models frequently has proven more exact and much less expensive as a solution to complicated problems for which a mathematical solution exists or for which a computer program could be established.

Structural models were used by Mr. Janney as design and analysis aids for five structures which have been built recently. Full scale tests were performed on three of these structures. The results of the model studies and the verification provided by the full scale tests were presented and discussed. These structures included a thin shell roof, folded plate roof, space frame roof, open spandrel arch bridge, and a precast bridge.

Full scale prestressed lightweight concrete experimental bridge

Truman R. Jones, Jr. and T. J. Hirsch, associate research engineer and assistant research engineer, respectively, Texas Transportation Institute, Texas A. and M. College System, College Station, Tex.

Presented the results of 3-year study of the field performance of a full scale, two span, precast, prestressed, multiple beam bridge. One span was of lightweight concrete and one span of sand-gravel concrete. Each span was identical in dimensions and composed of nine T-shaped members that formed the complete superstructure when assembled. The bridge was instrumented to furnish creep and shrinkage information. A comparison was made of the creep and shrinkage characteristics of lightweight and sand-gravel concrete for the full scale structure and for laboratory specimens. A procedure was presented for estimating creep and shrinkage stress losses in the field from the results of laboratory studies.

Failure of small reinforced concrete beams under repeated load

J. R. Verna and T. E. Stelson, respectively, teaching assistant and head, Civil Engineering Department, Carnegie Institute of Technology, Pittsburgh, Pa.

Ninety-six beams 72 in. wide, and 4, 5½, or 7 in. deep were investigated under repeated load. The effect of con-

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trolled variations in concrete strength and steel reinforcement resulted in four different modes of failure—concrete compression, diagonal tension, bond, and steel tension.

Twenty-four groups containing four identical beam specimens were studied. Of the four identical beams, one was tested to static ultimate load. For the remaining three the range of repeated load varied from a minimum of 10 percent to a maximum of from 40 to 85 percent of the measured static ultimate load.

The modes of static failure were compared with the modes of fatigue failure. The modes of fatigue failure were compared for different ranges of load. Relative safety for failure in different modes under repeated load and for failure in the same modes under static load were discussed as related to present design standards.

Fatigue properties of lightweight aggregate concrete

Warren H. Gray and J. F. McLaughlin, respectively, graduate assistant, Joint Highway Research Project, and associate professor of civil engineering, Purdue University, Lafayette, Ind.

This investigation was concerned with studying the fatigue properties of a lightweight aggregate concrete. An expanded shale aggregate was selected for test. Two mix proportions were used, one for a "low strength" concrete and one for a "high strength" concrete. Cylindrical specimens 3 in. in diameter and 6 in. high were cast. Many were tested in static compression to arrive at an estimate of the ultimate static compressive strength of each type of concrete while others were tested for fatigue in compression at stress levels of 40, 50, 60, 70, and 80 percent of the previously determined static ultimate strength. A statically sound analysis was insured by using over 40 specimens for the fatigue testing and over 60 specimens for the static testing.

The data from these tests on the various mixes were analyzed and compared. Comparisons were also made between the data obtained in this testing program and results from a similar study in which the fatigue properties of air-entrained and non-air-entrained normal weight concrete were investigated.

Prestressed composite design in multistory building construction

Harry Edwards, president, Leap Associates, Inc., Lakeland, Fla.

The pretensioned, precast concrete beam and joist is used to considerable advantage when combined as a composite structure with reinforced concrete floor and roof deck construction. The composite design offers considerable economy over a straight reinforced design. It also gives excellent camber and deflection control and permits long spans with heavy live loads with minimum depths of sections and with minimum of dead loads. Other items covered in the paper were fire ratings, connection details, composite action, fiber stress limits, and ultimate strength designs.

Prestressed girder design, testing, and construction for 21-story Norton Building, Seattle

Arthur R. Anderson, of Anderson, Birkeland & Anderson, Tacoma, Wash.

In the development of the design of the girders for this building, it was necessary to build a prototype and test it to ultimate strength. The interior girder (exterior frame of the building was of steel) featured nine large holes in the web of the I-section to accommodate mechanical and electrical installations. These large holes created some rather unusual design considerations. The full scale test was a dramatic demonstration of the ability of prestressed concrete to function on a long span to depth ratio and to allow

ACI President Joe W. Kelly (far left) points out some interesting features of University of Arizona architectural students' designs to three ACI members from Denmark (l. to r.): Ervin Poulsen, head of Departments of Structural Design and Applied Mathematics, Danish Institute of Technology, Copenhagen, Gunnar M. Idorn, director of research, Concrete Research Laboratory, Aktieselskabet Aalborg Portland-Cement-Fabrik, Karlstrup; and Poul Nerenst, manager, Research and Development Department, H. J. Henriksen and G. Kahler, Copenhagen

many large apertures to be located in the member.

Production of the precast members was illustrated as well as erection procedures.

Hyperbolic reinforced concrete cooling towers

Paul Rogers, Paul Rogers and Associates, and vice-president, Hamon, Inc., Chicago, Ill.

Thin shell, doubly curved, hyperbolic shaped, reinforced concrete cooling towers were offered as the ideal solution for the purpose of cooling and re-using industrial water to replace mechanical type cooling towers. Although some towers reached a height of 360 ft, with base diameter of about 300 ft, the wall thickness varied from 5 to 14 in. This is due to the extreme flexible strength and ability of doubly curved, thin shell construction.

Design and construction features were described briefly and the construction of one of these towers in Europe was illustrated.

Prismatic folded plates

A. A. Brielmaier, professor of civil engineering, Washington University, St. Louis, Mo.

The action of folded plates was explained with regard to the transverse slab action and with regard to the longitudinal plate action. The requirements for satisfactory supports were considered.

The analysis was considered from two viewpoints: the ordinary beam theory for rough preliminary designing and for reviewing, and the approximate folded plate theory which involves the determination of the shearing forces between the plates at their common edges. In the approximate theory, the effect of plate deflections on transverse moments and on edge shearing forces was taken into account. The principal plate or membrane stresses were then considered for each plate. The two methods of analysis and their results were compared by means of an example. The ultimate strength theory was briefly considered as a method for final flexural review.

Water vapor permeability of concrete

Donald F. Griffin and Robert L. Henry, Materials Division, U.S. Naval Civil Engineering Laboratory, Port Hueneme, Calif.

Basic data were presented about the water vapor permeability of plain concrete of varying water-cement ratios with equal consistency, and information was also given on the effects of certain admixtures such as oleic acid and sodium chloride on permeance. Oleic acid is of interest insofar as it reduces sack-hardening of cement; sodium chloride is of interest because sea water or brackish water may be the only mixing water available in tropical islands. A discussion was also included on salt whisker crystal growth on specimens with sodium chloride as an admixture.

Other variables included in the study were: (1) specific location of specimen disk as cut from a concrete cylinder; (2) maximum size of aggregate whether relatively large or small; and (3) environment of specimen whether in 73.4 F, 20 percent or 73.4 F, 50 percent relative humidity.

Part of the total study included the results of a quarter replicate statistical experiment for two levels of each of six factors to permit an analysis of variance of different variables in the premeance study. Data on standard deviations of compressive strength also were presented.

Looking Ahead to 1961 ACI Meetings

The 57th ACI Annual Convention will be held in St. Louis, February 20-23, at the Chase-Park Plaza Hotel. Join the hundreds of other ACI members who will converge for this 4-day meeting next month.

The 14th ACI Regional Meeting is scheduled for late October in Birmingham, Ala. The local planning committee is already shaping up program plans, not to be outdone by their fellow ACI members in Tucson.

Effect of steam curing on the important properties of concrete

Elmo C. Higginson, acting chief, Concrete Laboratory Branch, U.S. Bureau of Reclamation, Denver, Colo.

Investigations were conducted to determine the effect of steam temperatures from 100 to 160 deg, length of time of steam curing from 6 to 48 hr, and of a 1- and 3-hr delay prior to steaming, on the important properties of concrete. Properties of the concrete which were studied included compressive strength, durability as measured by the freezing and thawing test, permeability, volume change, resistance to abrasion, and resistance to sulfate attack. Research was also con-

Gene M. Nordby (far left), professor and head, Department of Civil Engineering, University of Arizona, Tucson (chairman of regional meeting organizing committee) emphasizes the good food being prepared for ACI while (center to right) Andrew Ross, professor of civil engineering, University of Arizona (vice-chairman of committee); J. Bruce Vesey, president, Construction Products, Ltd., Honolulu, Hawaii; and Samuel Hobbs, secretary-treasurer of ACI Southern California Chapter, Los Angeles, Calif., listen

Highlights of the meeting are discussed by (l. to r.) William S. Butcher, lecturer in civil engineering, Newcastle University College, Newcastle, NSW, Australia; Chih-Yuan Liu, Taiwan Water Conservation Bureau, Taipei, Taiwan; and ACI President Joe W. Kelly

ducted to determine steam curing effect on drying shrinkage, dynamic modulus, weight change to 6 months' age, and modulus of rupture.

In general, concrete that had been steam cured and then air dried was inferior to concrete that had been continuously fog cured or even fog cured for 7 days. Supplemental fog curing after the steam cure period improved the quality of the steam cured concrete. However, concrete of high quality can be produced using steam curing properly applied.

Concrete retempering studies

M. J. Hawkins, Concrete and Soils Control Branch, Central Valley Project, U. S. Bureau of Reclamation, Lewiston, Calif.

Transit-mixed concrete used by many contractors during construction operations on the Columbia Basin Project instigated a test program in the project laboratory to seek information about retempering. Test objectives were to determine the effects on concrete when mixing water additions were made to offset slump loss, involving long hauls or delays that resulted in extended mixing time.

Several test batches, mixed to about 3-in. slump and held in the concrete mixer for 6 to 8 hr were remixed and sampled at regular intervals during the mixing period. The concrete samples were tested for compressive strength and durability (freezing and thawing). The water-cement ratio, slump, and entrained air content of the concrete were also determined at each sampling period.

The retempering studies included tests to determine the evaporation losses of mixing water in concrete.

Properties of an expansive cement for chemical prestress

Alexander Klein, T. Karby, and Milos Polivka, University of California, Berkeley, Calif.

The preparation and properties of an expansive cement, essentially a calcium sulfa-aluminate clinker, and the characteristics of the concretes containing this cement were described. Such a cement is of major interest in the field of prestressing since it permits through chemical expansion the development of "self stress" in concrete.

Special concretes for missile launch pads

Herman G. Protze and Donald Garman, respectively, consulting technologist on concrete and construction materials, Boston, Mass., and Designed Concretes Co., Los Angeles, Calif.

Development of the interplanetary missile is changing not only the speed and distance of travel, but is also dramatically affecting the physical requirements of structures used in the launching of such vehicles. The paper considered the theoretical and prac-

TUCSON REGIONAL MEETING

The ACI 13th Regional Meeting in Tucson drew 354 registrants from 29 states, the District of Columbia, and several other countries. Registration records show seven members from outside the United States and twice that number were in attendance since in registering they used their U.S. address, being here for extended study. It was encouraging that 83 students were among the registrants. Here is a breakdown of the registration statistics.

UNITED STATES

Arizona	168	Ohio	2
California	63	Utah	2
Colorado	21	Wisconsin	2
Illinois	14	Delaware	1
Texas	11	Florida	1
New Mexico	9	Hawaii	1
Michigan	6	Iowa	1
New York	7	Kansas	1
Washington	5	Louisiana	1
Washington, D.C.	4	Massachusetts	1
Indiana	3	Minnesota	1
Missouri	3	Mississippi	1
New Jersey	3	Montana	1
Pennsylvania	3	South Dakota	1
Nebraska	2	Virginia	1
New York	7		

OTHER COUNTRIES

Denmark	3	Australia	1
Mexico	2	Norway	1

tical problems in the construction of semipermanent launching pads for missiles involving thrusts of 200,000 lb or greater and impingement temperatures up to 6000 F, utilizing liquid oxygen as the propellant.

Details were presented concerning a series of tests performed on experimental mixes of special concretes to determine their erosion rates (and life) in the exhaust flame of a LOX/RP-1, 1000-lb thrust vernier rocket engine. Materials, mixtures, equipment, and installation of the special concretes which have included normal portland cements, aluminous cements, and highly specialized structural refractory cements were described. Aggregates comprise special strong, heat-resisting types including semiprecious stones such as zirconium. Analyses of data obtained indicate that erosion rates decrease with increasing melting point of the cement. Also indications are that the erosion rate of any specific mix decreases exponentially with increasing distance from the missile engine exit.

A case of abnormally slow hardening concrete for tunnel lining

Lewis H. Tuthill, Robert F. Adams, Shelly N. Bailey, and Ronald W. Smith, respectively, concrete engineer and head of concrete laboratory, and engineers, California Department of Water Resources, Sacramento, Calif.

When 2 to 4 days elapse before concrete hardens appreciably beyond

First prize winner in the student architectural competition, Ray Krueger (right) receives the award from ACI Past President Frank Kerekes

initial set, it may, for all practical purposes, be considered abnormally slow in hardening. Retardation of several hours or half a day beyond normal may be purposefully induced for certain valid reasons in construction, but retardation for several days is abnormal and its unexpected occurrence can be most disturbing on a construction operation geared to safe removal of forms in 10 to 12 hr as in tunnel lining.

Heretofore, when concrete was slow in hardening, and if no admixture was used, it was usually due to: (1) concrete that was placed cold and stayed cold, although not necessarily frozen; (2) malicious or inadvertent inclusion of a considerable quantity of sugar or molasses; or (3) possibly some irregularity of the cement.

If an admixture capable of retardation was used, and concrete was much slower in hardening than usual, it was usually considered due to: (1) a gross overdosage of several times the normal amount of the admixture; or (2) crudity and variability of the product such that a material of exceptionally high sugar content or other retardant ingredient entered the concrete.

There is no record in such a case of the very slow hardening being ascribed to, or actually identified with, a sudden change in some aspect of the cement that was, nevertheless, well within the limits of ASTM requirements.

This report described a series of events, circumstances, and tests which was considered conclusive evidence that the grossly slow hardening which occurred with use of a lignin-based water-reducing retarder was due to a sudden reduction of the percentage of SO_3 in the cement from about 2.1 to about 1.6.

Epoxy resins as structural adhesives

James D. Kriegh and Terrill Ewbank, respectively, assistant professor of civil engineering and research associate, Department of Civil Engineering, University of Arizona, Tucson, Ariz.

Paper discussed the general use of epoxy resins as structural adhesives including their characteristics under various extreme conditions encountered in practice such as dry joints and joints made with wet to dry concrete. Tests on the specific use of epoxy resins as a shear connection in composite beams were reported. Also, a series of interesting tests on heavily damaged beams restored by use of epoxy resins was described.

Use of epoxy resins in highway field

R. E. Brown, Plastics Research Division, Shell Chemical Co., New York, N.Y.

The discussion pointed out, in a general way, how and where epoxy resins have been used with concrete, what their advantages are, and the kind of performance that may be expected. Some of the more prominent uses in highway construction have been skid resistant protective overlays; waterproofing membranes; patching compounds; crack and joint sealers; bonding agents for new to old concrete; general purpose adhesives for bonding traffic buttons, precast curbs, dowels, and other materials; and paints for highway and industrial uses. Examples

The design competition was a major feature of the exhibit area. Left—Gene M. Nordby (right), general chairman of meeting, talks it over with ACI President Joe W. Kelly. Right—Here, Gene Nordby (left) listens and Howard Harrenstien, student contest chairman, does the talking

of these applications to various projects were illustrated.

Application of the epoxies follows three major steps: (1) preparation of the surface, (2) blending of the components, and (3) placement and finishing of the blended components. Except for a few significant differences, for the most part the application methods and techniques do not differ widely from conventional concrete paving, patching, and bonding techniques. The second step, the blending of the components is perhaps the most critical part of the operation and generally represents the only radical departure from conventional techniques.

Committee meetings

In addition to the ACI Board of Direction and the ACI Technical Activities Committee meeting during the regional meeting, other administrative and technical committees scheduled sessions. The Board of Direction committees on chapter activities and long-range objectives met. The Technical Activities Committee on monographs met. Technical committees that scheduled either formal or informal sessions included Committee 208, Bond Stress; a subcommittee of Committee 212, Admixtures; the structural committee of Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete; the subcommittee on shear and diagonal tension, bond, and anchorage of Committee 318, Standard Building Code; ACI-ASCE Committee 326, Shear and Diagonal Tension; Committee 609, Consolidation of Concrete; and Committee 717, Practice in Low Pressure Steam Curing.

Student awards

The ACI organizing committee, in cooperation with the Civil Engineering Department and the Architectural School of the University of Arizona, sponsored a design competition for architectural students. The project called for design of a community for itinerate farm workers.

Judging was on the basis of appropriateness of use of concrete, imagination, functional planning, flexibility, economy, and total character of the community. The jury consisted of faculty members of the architecture and civil engineering departments of the university, a local architect, a local engineer, and a local concrete contractor.

First place and the $100 prize went to Ray Krueger. Second place and $30 prize was awarded to Paul Gumbinger; third place and $20 prize went to Al Burlini. Honorable mention certificates were given to Ralph Moore and Gerald Thayer.

The designs submitted in the competition showed imaginative and practical uses of concrete structures, with the use of precast elements predominant. Some designs included such

structural units as shells, folded plate roofs, lift slabs, and other forms.

The community was designed to provide quarters for itinerate seasonal agricultural laborers. Farm owners have found it desirable to contract for the harvesting of their crops with contractors who employ the laborers, house them, transport them to the fields, keep records, etc. Therefore, the design also called for housing a certain number of the contractor's staff.

Planning of the community and construction of the component buildings in concrete—mostly precast—had to be flexible for changing numbers of work force to be accommodated. The prototype design was based on 1000 laborers plus 30 supervisory and staff personnel.

Living units were to house mostly single laborers, with a modest number of units for families. Staff quarters were separate. Chapel, laundry facilities, kitchen, dining areas, store, toilet and bathing facilities, and a commons were included in the design.

Added attractions

The luncheon on Wednesday, November 2, was chairmanned by Past President Krank Kerekes, dean of faculty, Michigan College of Mining and Technology, Houghton, Mich. A. Allan Bates, vice-president of research and development, Portland Cement Association, Skokie, Ill. (president of American Society for Testing Materials and a director of ACI) was guest speaker at the luncheon. Student design competition awards were made by Dean Kerekes.

Added attractions planned by the local committee included a number of "extras" to keep the ladies well entertained while technical sessions were in progress. Tuesday, November 1 featured a luncheon-fashion show and a tour of Ash Alley shops, with ACI members and guests joining them in the evening for a social hour and western chuck wagon-style dinner. The following day the ladies enjoyed a bus trip into Mexico, luncheon at the Caverns dining room in Nogales, Sonora, and a shopping spree "south of the border."

Exhibitors

Exhibits were displayed by: Autolene Lubricants Co., Denver; Pneumatic Jack, Inc., Tucson; Rex-Spanall, Inc., Milwaukee; San Xavier Rock and Sand Co. and Deer-Williams Construction Industries, Inc., Tucson; Thomas Concrete Accessory Co., Los Angeles; E. A. Thompson Co., Inc., San Francisco; Waco Scaffolding and Equipment Co., Inc., Phoenix.

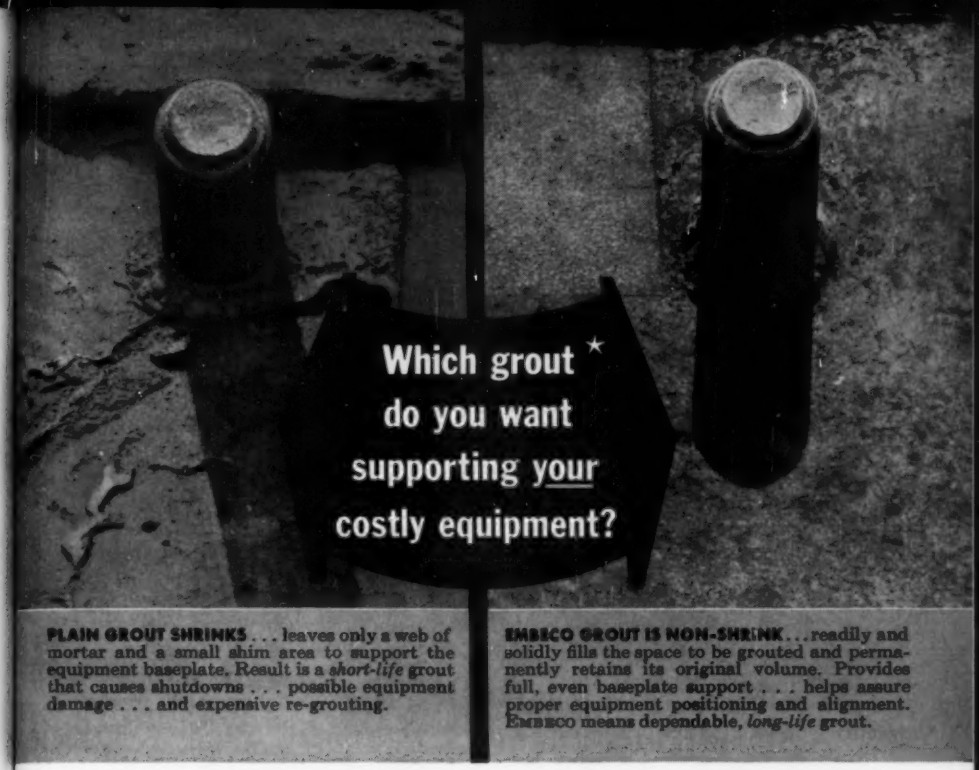
Organizing committee

Gene M. Nordby, professor and head, Department of Civil Engineering, University of Arizona, Tucson, was general chairman of the organizing committee. Andrew Ross, professor of civil engineering, University of Arizona, was vice-chairman and technical program chairman.

Edward Mangotich, general field engineer, Portland Cement Association, Tucson, was exhibits chairman, with Gene Anderson, civil engineer, Tucson, acting as sessions chairman.

Kenneth K. Kienow, research associate, Department of Civil Engineering, University of Arizona, headed up registration. Maurice LaGaard, associate professor of numerical analysis, University of Arizona, was in charge of field trips and Andrew Marum, Marum and Marum, Tucson, was membership chairman. Leland Lawrence, structural engineer, Friedman & Jobusch, Tucson, was in charge of tickets; Russell Taylor, Taylor - McFarland - Johnson, Tucson, was luncheon and banquet chairman; and Howard Harrenstien, associate professor of civil engineering, University of Arizona, was chairman of education contacts and student contests.

Mrs. Gene M. Nordby and Mrs. Maurice LaGaard were co-chairmen of the ladies' committee; assisting them were Mesdames Andrew Ross, Andrew Marum, Howard Harrenstien, and Edward Mangotich.



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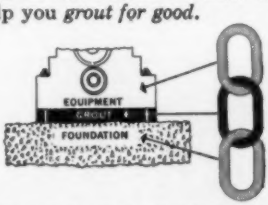

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Parabolic Arches and Rigid Frames Combined in Submersible Bridge

ONE OF THE LONGEST submersible bridges in the world to date has been completed over the Chambal River, Rajasthan, India.

The bridge was designed and constructed by Messrs. Gammon, India Private Ltd., under the direction of the chief engineer of the Central Public Works Department. The bridge is 2434 ft long between faces of abutments. The roadway level is nearly 90 ft above the river bed level in the central portion.

The bridge is practically an all weather crossing over the Chambal River on the Bombay-Agra Road, National Highway No. 3, near Dholpur. Although the normal high flocd level is 24 ft below the road level the maximum high flood level observed has been 27 ft above it. The bridge has therefore been designed to withstand stresses caused under such abnormal floods. Floods of this magnitude being unusual and of only a few hours duration, the bridge will actually be closed to traffic only seldom. A stretch of 700 ft of the river with a maximum depth of 26 ft is perennial.

The superstructure consists mainly of nine reinforced concrete parabolic arches over the deepest portion of the river and is flanked by 200 ft long reinforced concrete rigid frame units, three on one side of the river and four on the other. Four spans of 143 ft and five spans of 110 ft have been provided in the arched portion. The spandrel filling and piers consist of a 1:3 Colcrete mix with coursed rubble masonry, first sort, for the facing. The railing provided is of collapsible type.

The spans have been judiciously selected to give a balanced ratio between superstructure and foundations. The foundations are of widely different

Photo courtesy The Indian Concrete Journal

Superstructure of Chambal River Bridge consists of nine parabolic arches flanked by rigid frame units on either side

type as the river bed consists partly of exposed rock in slope and partly clayey soil overlaying rock and semi-rock formations. The piers for the arches rest on well foundations with Colcrete steining (stone or masonry lining) sunk to rock level up to a maximum of 70 ft; solid Colcrete cylinders replace the wells in the perennial stretch of the river. On one side the framed units rest on exposed rock and on the other side, about 14 ft below the ground level, on concrete foundations laid over clayey strata; to prevent scour of the bed during periods of high flood, the latter foundations for a length of bridge of nearly 800 ft have been provided with a Colcrete curtain wall on the upstream and downstream side founded deeper than the foundation level of the framed units on this side. In between the curtain walls is a Colcrete apron 18 in. thick and beyond for a length of 20 ft on the downstream side is a loose falling apron. The abutment piers are of reinforced concrete box type.

The bridge is designed for the usual Indian Roads Congress Class A and AA loadings.

Southern California Chapter conducts problem forum

ACI Southern California Chapter conducted a "Concrete Problems and Practice Forum" in conjunction with their November 17 meeting at the Rodger Young Auditorium in Los Angeles.

Program chairman Robert E. Tobin, Portland Cement Association, Los Angeles, as moderator put before the meeting a series of questions sent in as discussion subjects and called on "experts" and volunteers in the audience to furnish answers or comments. Answers and opinions were given and discussed freely, proving a fund of valuable information based on actual experience and technical knowledge.

Problems discussed in the forum included such subjects as: crack control in arid regions subject to critical atmospheric conditions and temperature changes; best methods and proper control of vibration for conventional and for lightweight concrete; use of coatings at perimeter of acid-stained pattern areas to produce clean-cut edges; methods and procedure for waterproofing concrete parking decks; jointing practice (or joint elimination) for slabs to be covered with asphalt tile; control of surface "bubbles" in concrete placed against plywood forms; causes and prevention of peeling and blistering on troweled concrete surfaces; procedure for determining strength development in prestressed concrete members.

In addition to the forum program, a series of selected slides was shown by David Leeds, professor, Department of Engineering, University of California at Los Angeles, illustrating the impressive modern concrete buildings constructed in the development of the city of Brazilia, new capitol of Brazil. Professor Leeds, who took the pictures during a recent visit, offered descriptive and explanatory comments as the views were shown.

A special guest of the chapter was Ahmed el Erian, associate professor at the University of Cairo, Egypt. Dr. Erian is in this country for a year of study and observation of engineering works and methods employed in concrete and other construction, under the program of the International Cooperation Administration.

Dundee Cement names Martin top executive

Dundee Cement Co., Dundee, Mich., has named Roblee B. Martin to the presidency, and to membership on the board. Mr. Martin has been chief resident executive officer since the company was founded 2 years ago.

Hans Gygi, who has been president of the concern since its inception, will become chairman of the board.

St. Louis, February 20-23

57th Annual ACI Convention

VERSATILITY KEYNOTES the outstanding 4-day meeting awaiting ACI convention-goers at the 57th Annual Convention next month in St. Louis at the Chase-Park Plaza Hotel. The technical program is geared to the interests of the designer, contractor, research man, and others in fields related to the concrete industry. Speakers of national and international prominence will present approximately 20 papers at five scheduled technical sessions.

The first 1½ days will be devoted to technical committee meetings with a Technical Committee Progress session on Tuesday afternoon, February 21. A number of brief committee reports will be presented on work underway and in progress within various committees.

The first general session opens Wednesday morning following addresses by Mayor Raymond Tucker of St. Louis and ACI President Joe W. Kelly, professor of civil engineering, University of California, Berkeley.

TECHNICAL PROGRAM

TUESDAY AFTERNOON, FEBRUARY 21

Technical Committee Progress Session

TAC Committee on Monographs—G. E. Burnett
Committee 115, Research—S. J. Chamberlin
Committee 116, Nomenclature—D. L. Bloem
Committee 212, Admixtures—B. E. Foster
Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete—T. R. Jones
Committee 318, Standard Building Code—R. C. Reese
Committee 321, Design of Reinforced Concrete Slabs—Joint ACI-ASCE—C. P. Siess
Committee 323, Prestressed Concrete—Joint ACI-ASCE—M. Schupack
Committee 335, Deflection of Concrete Building Structures — M. V. Pregnoff
Committee 616, Coatings for Concrete—W. H. Kuenning
Committee 621, Aggregates—W. R. Waugh
Committee 712, Precast Structural Concrete Design and Construction—Joint ACI-ASCE—J. R. Janney

WEDNESDAY MORNING, FEBRUARY 22

First General Session

President's Address—Joe W. Kelly, University of California, Berkeley

Report of Meeting of International Association of Bridge and Structural Engineers (Stockholm) — Douglas McHenry, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.

Report of Third International Congress of Precast Concrete Industry (Stockholm)—Arsham Amirikian, Bureau of Yards and Docks, Department of the Navy, Washington, D.C.

Report of Tours of Cement and Concrete Plants in Russia—A. Allan Bates, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.

Report of Autoclaved Cellular Concrete Conference (Gothenburg)—John K. Selden, Autoclave Building Products Association, Toledo, Ohio

WEDNESDAY AFTERNOON, FEBRUARY 22

Design and Construction Session

Prestressed and Preloaded Concrete Pillars—John J. Reed, Colorado School of Mines, Golden; and C. David Mann, St. Joseph Lead Co., Bonne Terre, Missouri

Mitchell Park Conservatory—W. J. Hufschmidt, Hufschmidt Engineering Co., Milwaukee

Analysis of Stresses and the Load Carrying Capacity of the Lincoln Boulevard Bridge over Lost River, Idaho—Reynold K. Watkins, William A. Cordon, and Elliot Rich, Utah State University, Logan

Supporting Structure for Retractable Roof of Pittsburgh Public Auditorium—Edward Cohen, Ammann and Whitney, New York; and H. Rey Helvenston, Public Auditorium Authority of Pittsburgh and Allegheny County, Pittsburgh

Materials Session

Water-Cement Ratio Versus Strength—Another Look—Herbert J. Gilkey, Iowa State University, Ames

Chemical Prestressing of Concrete Elements Using Expanding Cements—Alexander Klein and T. Y. Lin, University of California, Berkeley

How Good Is Good Enough?—E. A. Abdun-Nur, consulting engineer, Denver, Colo.

Load Tests on Patterned Concrete Masonry Walls—R. O. Hedstrom, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.

THURSDAY MORNING, FEBRUARY 23

Committee 216 Symposium on Fire Resistance

Introduction—Chairman C. C. Carlson, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.

Fire Resistance of Reinforced Concrete—Irwin A. Benjamin, Granco Steel Products Co., Granite City, Ill.

Fire Resistance of Building Construction Protected by Concrete—Robert R. Sheridan, Eastman Kodak Co., Rochester, N.Y.

Fire Resistance of Prestressed Concrete—G. E. Troxell, University of California, Berkeley

Design Session

A Controlled-Deflection Design Method for Reinforced Concrete Beams and Slabs—Adrian Pauw and Donald G. Alcock, University of Missouri, Columbia

Theory and Analysis of Prismatic Folded Plate Structures—John E. Goldberg and K. C. Mahmoud, Purdue University, Lafayette, Ind.

Rectangular Concrete Stress Distribution in Ultimate Strength Design—Allan H. Mattock, Ladislav B. Kriz, and Eivind Hognestad, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.

Tie Requirements for Reinforced Concrete—Boris Bresler and P. Gilbert, University of California, Berkeley

THURSDAY AFTERNOON, FEBRUARY 23

Research Session

Under auspices of Committee 115, Research

This session differs from general convention sessions in that the information is not released for publication. A group of short papers will be presented on research techniques and preliminary results on a number of current projects not yet sufficiently advanced to justify final reports.

Special events

Howard A. Coleman, vice-president and general sales manager, Missouri Portland Cement Co., St. Louis, is general chairman of the local committee, which is arranging an exhibit and a number of "extras" to make the 57th ACI annual meeting in St. Louis a highlight of the year.

Architectural student design competition

A design contest, with prizes totalling $400, will be held in conjunction with the convention, with competition open to architectural students at Washington University, St. Louis, and the University of Illinois, Urbana. The purpose will

be to recognize and reward significant contributions to structural design through more efficient use of concrete.

The design models will be displayed in the exhibit room at the convention.

The competition is open to any fourth or fifth year student enrolled in a recognized course of study of architecture at either university, provided he has taken part in the design and planning of the design submitted.

The entry must include a description of the structure by sketches, plans, section elevations and a model. The building to be designed is a chapel for an island naval base. It must be adaptable for services of Protestant, Jewish, Catholic, or interdenominational faiths and must have a capacity of 400 persons.

Judges for the contest will be one representative each from ACI, the faculty of Washington University, the faculty of University of Illinois, and a practicing architect from Illinois and one from Missouri. The first prize will be $250; second, $100, and third, $50. The judges reserve the right to withhold awards if they feel no entries deserve recognition.

Ladies' events

A full program of tours, luncheons, and other entertainment has been arranged for ladies at the convention. Included in the tours will be visits to St. Louis' famed waterfront, the Memorial Plaza, old and new sections of the city, the Climatron, Shaw's Gardens, and the Jefferson Memorial.

An Hawaiian breakfast at one of the city's largest department stores, a tour of another department store, and a luncheon at the convention hotel are scheduled. A dinner, Wednesday evening, February 22, will be preceded by a hospitality hour sponsored by the local committee.

In St. Louis, ACI members will find themselves at the traditional "Gateway to the West" with a colorful background unrivaled by any other city in the Midwest.

The St. Louis of today, which is Missouri's largest city and the nation's eighth largest, still takes pride in its varied history which weaves together the Indian lore of early America with the brave exploits of missionary explorers and the vision of colonial fur traders.

Today a hub city for transportation, manufacturing, commerce, and finance as well as the dominant center in a wealthy agricultural area, St. Louis is the true product of its past, combining a pride in its historic heritage with the spirit of progress which made it great.

Situated in the middle of the nation, St. Louis, the city of a thousand sights, is considered an ideal convention city.

Plan to attend the 57 Annual ACI Convention, February 20-23 at the Chase-Park Plaza Hotel, St. Louis.

EXHIBIT

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•

FEB. 20-23, 1961
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ACI Exhibits Chairman
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Prestressed Footing Rings for Missile Base

A MAMMOTH 1600 MILES of steel wire, under tension of 150,000 psi, were wound around the circular footings supporting 12 igloo-shaped power houses and control centers at the Titan missile base at Lowry Air Force Base, Denver. Up to 25 layers of prestressing wire were applied to the footings to counteract normal loading and potential nuclear blast forces.

The base includes six underground (the first) missile-launching "complexes," widely separated from each other. Each complex contains a power house, a control center, three missile-launching silos, two antenna silos, and interconnecting tunnels.

The hemisphere-like shells housing the power facilities are of reinforced concrete with reinforcing steel anchored in the footings. This reinforcing steel, together with footing keyways, transfers stresses induced by normal loading and possible blast loading to the circular footings.

To counteract these stresses, high-tensile wire under tension was wound around the circumferences of the footings after completion of the structures.

Wire-winding machine being readied to prestress footing of missile base structure. Machine will wrap multiple layers of wire around circumference of footing to withstand normal and blast loading

Twenty-five layers of wire, each layer containing 98 wires, were applied to impart the required prestressing force. As each layer of wire was placed, it was covered with pneumatic mortar, and a final layer (⅝ in.) of mortar was applied to the completed job.

Two wire-winding machines designed especially for this job applied the wire. Each self-propelled machine was fitted with one pair of wheels to run along the top of the footings, and another set to run along the outside face. As the machine traveled around the footing, the prestressing wire was drawn through a die from a diameter of 0.192 in. to 0.172 in. to apply the required tension. The upper portion of the machine was guyed to a circumferential wire secured to the power house structure part way up from the base. The machine propelled itself by driving a rotating member which meshed with a circumferential chain secured at the base of the structure.

Structures similar to, but of slightly smaller dimensions, house the control center facilities. Prestressing was carried out in the same manner as at the power houses, but because of different loading requirements, only 11 layers of wire were required.

The general contractors were Morrison-Knudsen Co., Inc. and Associates. Prestressing was by The Preload Co., Inc.

Soiltest appoints Manatt to international sales post

Soiltest, Inc., Chicago, has announced the appointment of Robert N. Manatt as director of international sales.

Mr. Manatt is a graduate of The College of William and Mary, Williamsburg, Va. and the American Institute for Foreign Trade, Phoenix. He was formerly on the staff of Iowa Wesleyan College, Mount Pleasant.

Positions and Projects

ACI expands service to members

ACI has re-emphasized its policy of taking the Institute to members by creating the staff position of assistant secretary with primary responsibility to develop chapter activities, to keep step with the continuing expansion of the Institute.

The new staff position of assistant secretary, authorized by recent action of the ACI Board of Direction, has been assumed by Robert E. Wilde who was formerly managing editor of the JOURNAL. In announcing the new staff position, ACI President Joe W. Kelly also disclosed the promotion of Robert G. Wiedyke to managing editor and the appointment of George D. Nasser as assistant editor.

The new post of assistant secretary, President Kelly explained, was necessitated by ACI's steady membership growth and broadening services. More than 10,300 persons in 93 countries who have a professional interest in concrete are members of ACI. The monthly JOURNAL, special publications and handbooks, and technical meetings and conferences are the principal mediums of exchange of concrete information among ACI members. Increased chapter activity was indicated as the next phase of expanding this exchange of information.

In addition to helping expand the chapter program (ACI has four chapters in the United States now), Mr. Wilde will act as principal assistant to the Institute's secretary-treasurer, William A. Maples. He will oversee the management of national and regional technical meetings and supervise the development of staff systems and procedures at ACI headquarters. Mr. Wilde also will work closely with Kenneth D. Cummins, ACI technical director, and with the Technical Activities Committee to develop information covering a wide range of subjects in the field of concrete technology.

Mr. Wilde steps into the position of assistant secretary thoroughly familiar with ACI policy and procedures and well indoctrinated in the staff philosophy of service to members. He is a 1949 civil engineering graduate of Iowa State University, Ames. He first joined ACI as associate editor of the JOURNAL in March, 1949, remaining until July, 1951, when he moved to Chicago to become associate editor of *Rock Products*. He returned to ACI in October, 1952, to become managing editor of the JOURNAL.

Robert E. Wilde

Mr. Wilde is a member of the American Society of Civil Engineers and Sigma Delta Chi, professional journalistic society. He is active on the Concrete Improvement Board of Detroit, as well as U. S. Army Reserve, Corps of Engineers.

Mr. Wiedyke, formerly associate editor of the JOURNAL, is a civil engineering graduate of Wayne State University, Detroit. He joined ACI as assistant editor in June, 1954. From June, 1956, to June, 1958, he served with the 538th Engineer Battalion (Construction) at Fort Knox, Ky. He has been associate editor since returning to the staff in 1958. Mr. Wiedyke is a member of ASCE and is currently working on his MS in civil engineering

Robert G. Wiedyke

George D. Nasser, newly appointed assistant editor, was born in Addis Ababa, Ethiopia. He received his early education in Cyprus, Kenya, and Eng-

George D. Nasser

land. Returning to Addis Ababa, he earned an Intermediate Science degree at Haile Selassie's University College in 1955.

That same year a tennis scholarship brought him to the United States where he enrolled at the University of Detroit. He was awarded his BS in civil engineering in June, 1958. Mr. Nasser attended graduate school at Purdue University (highway engineering) and the University of Michigan (highway transportation), receiving his MSE from the University of Michigan in June, 1960. He is a member of ASCE.

Kenneth D. Cummins, ACI technical director, completes the present administrative and technical staff at Institute headquarters. Thomas C. Akas is advertising manager of the JOURNAL and head of membership department.

Mission revised for ACI Committee 609

The Board of Direction recently approved the following revised mission for Committee 609, Consolidation of Concrete: The mission of this committee is to encourage research of the basic effects within concrete produced by consolidation devices of different design and capacity; to promote the development of approved devices and practices; to evaluate and to disseminate such knowledge in the field of concrete.

Proceedings volume to be changed to coincide with calendar year

The Board of Direction approved a Technical Activities Committee recommendation that the ACI *Proceedings* volume be changed to coincide with the calendar year beginning in 1961. The current volume (*Proceedings* V. 57) will close with the June, 1961, JOURNAL. *Proceedings* V. 58 will run from July to December, 1961, and *Proceedings* V. 59 and volumes thereafter will begin with the January isues of the JOURNAL each year.

The Board also approved the deletion of the JOURNAL volume number on the next volume beginning July, 1961.

Board approves revision of Conduct of Committee Work

On the recommendation of the Technical Activities Committee, the Board of Direction recently approved an addition to Section 1, Technical Committees, in the ACI "Conduct of Committee Work:" *Members of the technical committees of the American Concrete Institute are selected on the basis of the personal knowledge they can contribute to committee activities and the American Concrete Institute. Although much benefit is often derived from the support given to a committee member through his affiliation with an organization, firm, or agency, it is not the intent that committee membership be on the basis of agency or firm representation.*

Board approves revision of Publications Policy

The Board of Direction has approved revisions in the "ACI Publications Policy" as recommended by the Technical Activities Committee.

The revisions clarify the evaluation procedure employed in screening material for publication in the ACI JOURNAL and other ACI books and bulletins. The various categories into which contributions may be classified are discussed. These include: JOURNAL papers, Concrete Brief articles; Current Reviews, bibliographies, special publications, chapter publications, and papers suitable for presentation at annual and regional meetings.

In updating the Publications Policy the Technical Activities Committee has

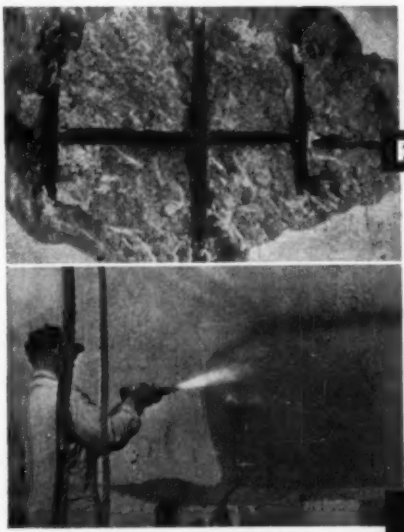
gone into greater detail to explain the basis of acceptability. The guide section on preparation of manuscripts, tables, and illustrative material, remains unchanged. The complete policy will be printed in the *1961 ACI Directory*, to be published this spring.

ACI Michigan Chapter elects officers

William C. Krell, supervising structural engineer, Michigan District, Portland Cement Association, Detroit, was recently elected president of the ACI Michigan Chapter. Richard H. McClurg, president, R. H. McClurg Associates, Inc., Detroit, was elected vice-president; A. J. Mullkoff, civil engineer, Concrete Steel Corp., Detroit, was elected secretary-treasurer. Six directors were elected: Daniel S. Ling, professor, College of Engineering, Wayne State University, Detroit; and Joseph A. Kauer, technical service director, Huron Portland Cement Co., Detroit, for 3-year terms; Vance J. Gray, engineer, Raymond C. Reese, Toledo, Ohio; and Leland V. Maschmeyer, vice-president, Maschmeyer Transit Mix, Roseville, Mich., for 2-year terms; James E. Hampton, technical director, American Aggregates Corp., Detroit; and Howard L. Lilly, city engineer, City of Dearborn, Mich., for 1-year terms.

While the group has been meeting informally for close to 2 years, a formal petition for the chapter was accepted by the Board of Direction at the time of the 1960 annual convention in New York City. The Michigan Chapter embraces all of the state of Michigan and Lucas County of Ohio, where the principal city is Toledo.

The chapter now has about 70 members and holds monthly luncheon meetings. E. M. Wheat, chief structural engineer, Eberle M. Smith Associates, Inc., architects and engineers of Detroit was speaker at the November 9 meeting. He spoke on "Design Techniques Used in Double Sine Curved



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Roof," augmenting his talk with slides showing the development of the idea, results of the model tests, and construction of the formwork. Pictures of the site, model, and architect's rendering depicted the desired final result.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 331, Structures of Concrete Masonry Units

Paul M. Woodworth
The Waylite Co.
Chicago, Ill.

Committee 612, Recommended Practice for Curing Concrete

S. B. Helms
Lehigh Portland Cement Co.
Coplay, Pa.

Committee 616, Coatings for Concrete

Charles A. Vollick
Sika Chemical Corp.
Passaic, N.J.

Mather named chairman of ASTM Committee C-9

Bryant Mather, supervisory civil engineer and chief, Special Investigation Branch, Concrete Division, U. S. Army Engineer Waterways Experiment Station, Jackson, Miss., has been named chairman of ASTM Committee C-9, Concrete and Concrete Aggregates. He was formerly secretary of the committee. Mr. Mather succeeds Walter H. Price, chief research engineer, Engineering Laboratory, U. S. Bureau of Reclamation, Denver.

An ACI member since 1944, Mr. Mather is currently a member of the Institute's Board of Direction. He has served three terms as chairman of the ACI Technical Activities Committee and presently is a member of that committee. He is also a member of Committee 116, Nomenclature; Committee 212, Admixtures, and Committee 612, Recommended Practice for Curing Concrete.

Poulsen visits ACI headquarters

Ervin Poulsen, head of Departments of Structural Design and Applied Mathematics, Danish Institute of Technology, Copenhagen, Denmark, recently spent 2 days at ACI headquarters.

Professor Poulsen has been observing American construction techniques and inspecting outstanding buildings, particularly shell structures. He has taken some 600 photographs to be shown with subsequent lectures in Copenhagen.

One of the prime purposes of Professor Poulsen's trip to the United States has been to discuss with educators, present-day methods of teaching design of shell structures to engineering students. He himself is an advocate of the ultimate strength design method and use of models, a method apparently not widely used in engineering education in the United States.

Kalve joins Chicago architectural firm

Ernest Kalve has joined Schmidt, Garden and Erikson, architects and engineers, Chicago, as a structural engineer.

McCoe joins Permanente technical staff

Jack C. McCoe has been appointed assistant technical director of Permanente Cement Co., Oakland, Calif.

Prior to joining Permanente, Mr. McCoe held the position of technical director of the cement division of Pittsburgh Coke & Chemical Co., with whom he was associated since 1953. In his new position with Permanente Cement, he will headquarter in the technical laboratory at the company's plant near San Jose, Calif.

Detroit to host NCMA 41st annual convention

The National Concrete Masonry Association's 41st annual convention and 12th Concrete Industries Exposition will be held in Detroit, January 30-February 2. Location for the exposition and all convention meetings will be in Detroit's ultramodern new Cobo Hall. This convention center is said to be the world's largest exhibition building.

Ralph W. Walb, General Dredging Co., Fort Wayne, Ind., is program chairman for the meeting. The program will cover engineering and research activities, aspects of plant management, advertising and sales promotion, and briefings by top-level officials of the federal government's housing and civil defense agencies.

Jolly named superintendent of Huron's Alpena plant

John W. Jolly has been named general superintendent of the Huron Portland Cement Co. plant in Alpena, Mich. Mr. Jolly has been assistant to the mill manager for the past year. In his new position Mr. Jolly succeeds Elmer Evans, who has been named superintendent of construction.

Billington joins Princeton faculty

David P. Billington, formerly project manager, Roberts and Schaefer Co., New York, is now an associate professor of civil engineering at Princeton University.

Professor Billington graduated from Princeton University in 1950 with a BS in engineering. For the academic year, 1950-51 he won a Fulbright Fellowship to Belgium in structural engineering, where he combined studies at the University of Touvain with travel through Belgium and France to learn of European advances in structural design since World War II. As a result of this work he was granted a renewal of his fellowship for the following year to study under Prof. Gustave Magnel at the University of Ghent, Belgium, and prestressed concrete in Professor Magnel's laboratory.

Professor Billington has been an Institute member since 1953 and is currently serving on ACI Committee 339, Allowable Stresses in Reinforcement, and ACI-ASCE Committee 712, Precast Structural Concrete Design and Construction.

Errata

The following corrections should be made in "Resistance to Shear of Reinforced Concrete Beams. Part 5—Anchorage and Bond," by J. Taub and A. M. Neville, which appeared in the December 1960 ACI JOURNAL.

p. 715—in the notation change o to Σo.

p. 721—in the last sentence of the second paragraph change 190 to 1.90.

p. 723—in the ninth line from the bottom of the page change Σ_o to Σo.

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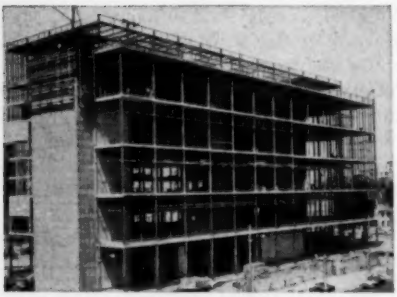

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J-11

Alpha Cement appoints Heineck to new post

Dale W. Heineck has been appointed manager of Alpha Portland Cement's newest plant at Lime Kiln, Md. He succeeds George V. Cosgrove who has been promoted to manager of process engineering at the company's Easton, Pa., home office.

Mr. Heineck, formerly assistant manager of Alpha's LaSalle, Ill., plant, has been with Alpha since 1954.

Raphael elevated to full professorship

Jerome M. Raphael, formerly associate professor of civil engineering, University of California, Berkeley, is now a full professor at the university. He joined the faculty in 1953.

Professor Raphael has been an Institute member since 1934 and is currently serving on ACI Committee 207, Properties of Mass Concrete, and Committee 209, Volume Changes and Plastic Flow in Concrete.

Rosenberg joins Dewey and Almy

Arnold M. Rosenberg has joined Dewey and Almy Chemical Division, W. R. Grace and Co., Cambridge, Mass., as senior research chemist in the construction chemicals and rock products chemicals laboratory. Dr. Rosenberg will be engaged in basic research in the hydration of portland cement and the effect of admixtures on concrete strength and quality.

Universal Atlas Cement plans more distributing stations

Universal Atlas Cement, New York, a division of U. S. Steel Corp., has secured options on three sites for cement distributing stations along the eastern seaboard. The sites selected are at Norfolk, Va., and Jacksonville and Port Canaveral, Fla.

Composite Construction Converts Old Bridge to "New" One

By V. M. ROMINE*

THROUGH THE USE of composite construction during the replacement of a concrete slab roadway, an Illinois bridge was recently converted to provide a greater load carrying capacity at little additional cost.

The 1500-ft Burr Oak Avenue Viaduct in Blue Island, Ill., was upgraded with composite design from an AASHO load rating of H18-S15 to the equivalent of H20-S16.

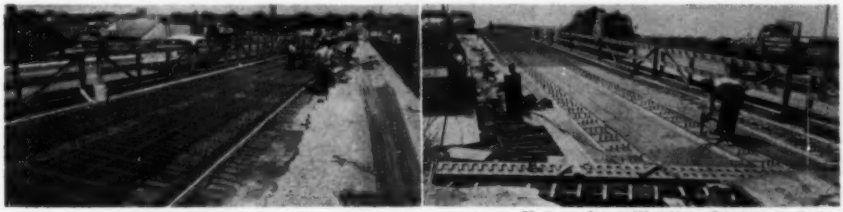
The prime reason for reconstructing the bridge was the deterioration of the 7 in. thick roadway, although undesirable vibration and live load deflection were also factors in deciding to rebuild. Steelwork was considered structurally adequate. By bonding the new concrete slab to the steel beams, vibration and deflection were considerably reduced.

Since the concrete had to be replaced, composite construction was considered the most economical method of stiffening the structure. If it had not been necessary to replace concrete, it probably would have been more economical to upgrade by welding cover plates to the bottom flanges of the beams. Cover plates would have increased steel stiffness but probably would not have reduced vibration as much as composite construction. Under the circumstances, no cover plates were used.

Nelson headed shear connector studs 4 in. long and ⅞ in. in diameter were used. A total of 24,160 of these studs were end welded to the upper flanges of the bridge girders with a stud welding gun. Studs were welded four in a row with rows approximately 6 to 18 in. apart, and were installed at a rate of about three studs per minute. In addition to the rapid speed of placement and the elimination of distortion, which may result with prolonged arc welding time, the use of end-welded studs simplified installation of the shear connectors near rivet heads on the plate girder spans.

Traffic was maintained on one-half of the bridge, while the other half was being rebuilt. General contractor was States Improvement Co., Maywood, Ill.

*Engineer of Bridge and Traffic Structures, Illinois Division of Highways, Springfield, Ill. (Deceased)



Nelson Stud Welding Division photos

When deteriorated concrete slab had to be replaced on the Burr Oak Avenue Viaduct, Blue Island, Ill., state engineers also raised the AASHO load rating by converting the structure to composite construction. End welded studs were chosen as shear connectors. These two views show the installation of shear connectors and reinforcement

LOOKING AHEAD

Jan. 9-11, 1961—Annual Convention, Concrete Products Manufacturers Association of Quebec, Chateau Frontenac, Quebec City, Que., Canada

Jan. 9-13, 1961—Annual Meeting, Highway Research Board, Washington, D.C.

Jan. 17-19, 1961—44th Annual Convention, National Crushed Stone Association, Hotel Americana, Bal Harbour, Fla.

Jan. 17-19, 1961—Winter Conference and Exhibit, Instrument Society of America, Sheraton-Jefferson Hotel and Kiel Auditorium, St. Louis, Mo.

Jan. 18-20, 1961—Annual Convention, National Concrete Contractors Association, Diplomat Hotel, Hollywood-by-the-Sea, Fla.

Jan. 23-26, 1961 — Annual Plant Maintenance and Engineering Show, International Amphitheatre and Palmer House Hotel, Chicago, Ill.

Jan. 23-26, 1961—31st Annual Convention, National Ready Mixed Concrete Association, Hotel Americana, Bal Harbour, Fla.

Jan. 26-28, 1961—13th California Street and Highway Conference, Institute of Transportation and Traffic Engineering, and University Extension, University of California, Berkeley, Calif.

Jan. 30-Feb. 2, 1961—12th Biennial Concrete Industries Exposition, National Concrete Masonry Association, Cobo Hall, Detroit, Mich.

Jan. 30-Feb. 3, 1961 — Committee Week, American Society for Testing Materials, Netherland Hilton Hotel, Cincinnati, Ohio

Feb. 20-22, 1961—Annual Convention, National Concrete Products Association, Seigniory Club, Montebello, Que., Canada

Feb. 20-23, 1961—57th Annual Convention, American Concrete Institute, Chase-Park Plaza Hotel, St. Louis, Missouri.

Mar. 13-16, 1961 — 53rd Annual Convention, American Concrete Pipe Association, Hotel Americana, Bal Harbour, Fla.

May 15-16, 1961 — 41st Annual Meeting, Society of American Military Engineers, Washington, D.C.

Design of Concrete Pavements

Recommended Practice for Design of Concrete Pavements (ACI 325-58) covers the design of rigid airport and highway pavements and bases for conditions of climate, traffic, available construction materials and equipment, and construction methods of the United States. Includes recommendations for soil foundations, selection of slab dimensions, joints, and details for reinforced or nonreinforced concrete. 36 pp. $1.00 per copy, 50¢ to ACI members.

Order from Publications Department, American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Michigan.

Walter C. Voss

Walter C. Voss, formerly head, Department of Building Engineering and Construction, Massachusetts Institute of Technology, Cambridge, died recently at the age of 73. Professor Voss, an ACI member since 1917, was one of the early pioneers of the Institute.

Born in Chicago in 1887, he graduated from Chicago Teachers College in 1907, the University of Illinois in 1912, and MIT in 1932. After 2 years work in structural design in Chicago, he became head of the Department of Architectural Construction at Wentworth Institute in Boston in 1914. Professor Voss served as district structural engineer with the Portland Cement Association from 1925 until he joined the MIT faculty in 1928.

Professor Voss was widely known as a consultant in architectural construction and materials. He authored a number of books and countless technical articles. In addition to his interest in ACI, Professor Voss was active in the executive and technical committee work of the American Institute of Architects, American Society for Testing Materials, American Standards Association, and the Building Research Advisory Board.

Lyon moves to Japan for Master Builders

Charles A. Lyon is now headquartered at Nisso Master Builders, Tokyo, Japan.

A veteran of almost 20 years with Master Builders, he joined their merchandising staff in 1941. He served as manager of the Detroit branch office prior to his appointment last year as special assistant to the vice-president for marketing, working out of the Cleveland office.

An ACI member for the past 10 years, Mr. Lyon has been active in

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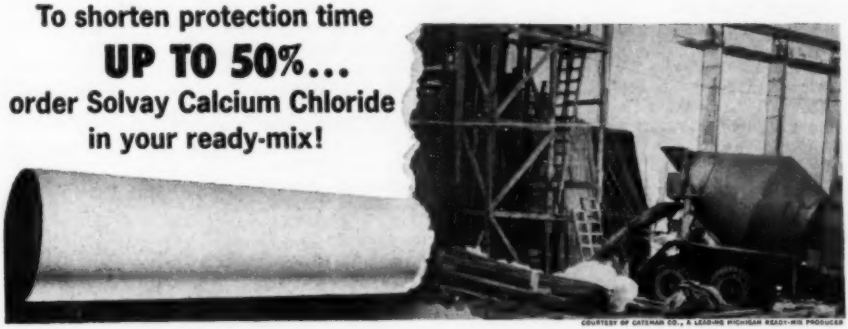

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committee work in the Institute, Concrete Improvement Board of Detroit, Construction Specification Institute, and the Michigan Ready-Mixed Concrete Association.

Klein and Smith retire from Dragon Cement staff

With a combined total of 101 years of service in the cement industry, W. H. Klein, executive vice-president, production, and R. H. B. Smith, executive vice-president, sales, Dragon Cement Co., Thomaston, Me., recently retired from their posts with this American-Marietta Co. division.

Mr. Klein has been active in ACI activities since 1924. He was a member of the Institute Board of Direction in 1947-48 and served on the Publications Committee from 1943-46.

Both Mr. Klein and Mr. Smith are widely known in their respective fields and both have been active in supporting the work of industry and technical societies in New England and the Middle Atlantic states.

Honor Roll

Jan. 1—Nov. 30, 1960

Have you signed a new ACI member this month? Among your acquaintances working in concrete, there are probably several excellent ACI prospects who would be only too anxious to join if they knew more about ACI services. Why not tell them about ACI today and have your name listed in next month's honor roll.

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1 point for Student; 2 points for Junior; 3 points for Individual; 4 points for Corporation; and 5 points for Contributing.

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Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. All who have an interest in concrete are eligible for membership. The grades of membership are described below.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI Journal provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings, they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

ACI's world-wide membership is growing in extent and participation —traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

Individual Members (U.S. and Possessions, Canada, Mexico, Central America, and West Indies)	**$20.00**
Individual Members (All other foreign countries)	**16.00**
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Board of Direction, American Concrete Institute
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The undersigned hereby applies for admission to the American Concrete Institute as ☐ *Individual* ☐ *Corporation* ☐ *Contributing* ☐ *Junior* ☐ *Student Member and agrees to be governed by the Charter and Bylaws.*

Name and complete mail address of proposed membership (Address to which Journal is to be mailed—please letter)_____

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For Corporation Membership, ACI representative will be _____

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For our records, please complete both sides of application.

NOTES on Membership Classification

The ACI advertising and editorial departments need definite information concerning job title, classification, business affiliation, and principal responsibility of all members. Thus, the information requested on the classification form below is especially important.

The applicant should designate his title or position such as: construction superintendent, research director, technician, draftsman, structural engineer, inspector, plant superintendent, highway engineer. The occupation of the applicant should be that which is **most frequently** performed. The principal responsibility of the applicant is that specific area of his job for which he is **primarily concerned.**

Classification is not based on what interests you, but on what you do—what office or job you fill. When completing the form below, please check the one item in each section that is most applicable to that particular section.

ACI editors want to know your interests—they welcome suggestions as to what you'd like to see discussed in the ACI Journal and of the possible sources. Please attach a separate note or letter.

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The Board of Direction approved applications in the following categories: 72 Individual, 20 Junior, and 32 Student, making a total of 124 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership now stands at 10,347.

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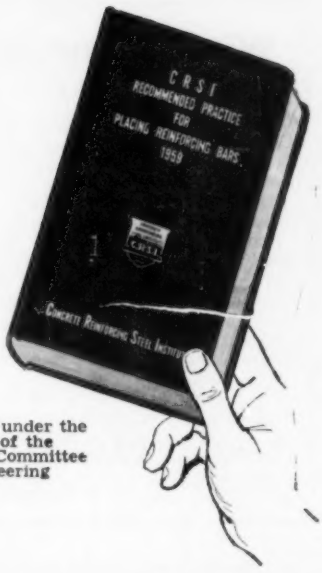

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RECOMMENDED PRACTICE FOR WINTER CONCRETING (604-56) advises the use of air-entrained concrete and addition of one percent of calcium chloride by weight of cement for cold weather concreting. These permit a reduction in the time newly placed concrete should be protected. Other factors covered in this ACI Standard include the use of accelerators and antifreezes, keeping of temperature records, heating of materials, subgrade preparation, protective coverings, heated enclosures, curing and form removal. 23 pp. Price 75¢, 50¢ to ACI members.

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Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Photo-recording of highway surfaces

The Sur/Fax system of photo-recording highway surfaces has been announced by Aero Service Corp. The system includes a precise strip camera, a specially equipped truck, a film projector, and a reader-printer.

The new system has performed successfully at the AASHO test area in Ottawa, Ill., and a program for coverage of major test areas in New York is in progress.

The truck unit carries a stabilized, vibration-free continuous strip camera. It records highway surfaces as wide as 36 ft in a single traverse, on 35-mm film, at a scale of 1 in. equals 36 ft.

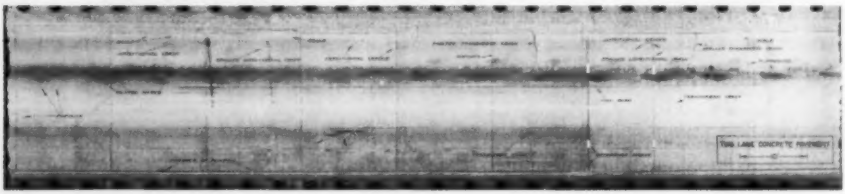
The truck also mounts a battery of shielded, high-powered lights. These provide a constant level of light, beamed at a constant angle to obtain just the right degree of light and shadow. Under this controlled lighting system, such conditions are faulting, scaling and cracking are readily identifiable.

Surveys are made at night at 20-30 mph, generally between 1 a.m. and 5 a.m. to avoid traffic and daylight. (Sunlight at varying degrees of intensity and changing angles to the highway surface, will not provide the constant light source needed for these surveys.) A generator in the rear of the truck provides power for lights and camera.

The continuous strip film positives may be projected as large as 10 x 10 ft for staff conference studies. With the projector, the film may be viewed at a rate equivalent to slow walking speed, about 2 mph. The adjustable-speed driving mechanism can be stepped up to an equivalent of 30 mph or it can be stopped and a film section "held" on the screen for detailed examination.

For detailed analysis, a specially modified Minnesota Mining and Manufacturing Corp. reader-printer is used. The 35-mm film strip is projected onto an 18 x 24 in. viewing screen. As pavement sections come into view which require continued study, a touch of the push button stops the film and makes a positive print. This print is delivered within 30 sec. It may be annotated and used as an actual work sheet by maintenance crews.

Sur/Fax record forms are provided for engineers and technicians to note the data seen on the continuous strip film. For rigid pavements, such facts include: cracks of all types, scale, spalling, and faulting. For flexible pavements, the recorded conditions in-



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clude: alligator-type or other cracks, raveling, rutting, settlement, and bleeding.

According to Aero Service engineers, these data can be transferred to IBM punch cards for statistical analyses and broad studies of highway conditions over large areas.

Other applications are seen for this new photo-recording system. Tests are under way for use in inventorying railroad or subway roadbed, bridges, and airport pavements.—Aero Service Corp., 210 E. Courtland St., Philadelphia 20, Pa.

Metal corner guard for forms

Gates & Sons, Inc., has introduced a new metal corner guard for use with its plywood form panels. Constructed of malleable iron, the guards are flush-mounted to the panel forms with rivets provided.

By reinforcing the area of the panel most subject to deterioration, it is said that the corner guards will provide longer, more economical use of form panels.—Gates & Sons, Inc., Denver, Colo.

Truck crane

A 20-ton capacity truck crane has been developed which is said to safely handle booms and jib extensions up to 140 ft in length.

Technically labeled a P&H Model 255B-TC truck crane, when equipped with a maximum 110-ft boom and 30-ft jib extension, the crane

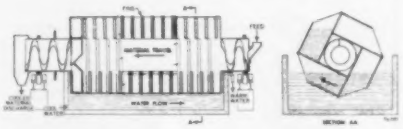
has proved capable of delivering a 5500-lb load 12 stories about ground, or positioning 4300 lb loads 60 ft away.

Expected to be of particular interest to small contractors is the fact that the unit can be equipped with a dragline bucket, clamshell bucket, or pile driver, all using the standard 40-ft boom used for hoisting work.

Advanced engineering design has resulted in an additional bonus by the way of the machine's ability to travel public highways. Convenient removal of certain components automatically provides an axle weight distribution as acceptable as most 15-ton truck cranes on highways of any state. Highway speed was reported to be a maximum of 40 mph.—Harnischfeger Corp., 4444 W. National Ave., Milwaukee 16, Wis.

Conduction-type drum cooler

The Roto-Fin cooler, a basically new conduction-type drum cooler which can handle fine materials at high temperatures and cool



them rapidly and efficiently, has been announced by Link-Belt Co.

The unit is a rotating drum with a series of deep, hollow, watertight fins or "cells" which are lapped consecutively around the periphery of the drum. The drum revolves partly submerged in water; the material to be cooled is conveyed down its spiral length from one turn in the spiral to the next. The extremely large conduction cooling surface provided by this design assures rapid heat transfer, and a relatively short drum is required according to company spokesman. The drum of the cooler can be any length and diameter, depending on the plant layout, design capacity, and amount of heat transfer required.

The cooler has a minimum of moving parts. No fans, dust collectors, or controls are required. It is designed to be self-cleaning, with convenient feed and discharge locations.—Link-Belt Co., Prudential Plaza, Chicago 1, Illinois.

Grouting technique for gas compressors

A resin-based grout, Ceilcote 648, is a quick-setting grout specially compounded to provide a high-strength foundation that is virtually impermeable to oils, most acids, and alkalies. Other important physical properties cited by Ceilcote include a tensile strength of 1950 psi and a compression strength of 15,000 psi. It is said to flow smoothly around complex configuration of compressor bases without shrinking or settling to provide a singularly high grout-to-metal bond of 1000 psi.—The Ceilcote Co., 4832 Ridge Road, Cleveland 9, Ohio

Flooring and patching compound

E-Bond #1005 is an epoxy-based compound that welds itself to concrete flooring or any masonry surface and gives a bond with four times the abrasion resistance of concrete according to the developer. It can be applied to old or new concrete, wood, ferrous and nonferrous metal.

The material is said to have remarkable resistance to acids, alkalies, and solvents; also it will not break down under constant wear and impact. It can be pigmented in a wide range of colors; trowels to a minimum of 1/16 to 1/4 in. and can be feather-edged.—Dr. R. E. Robinson Laboratory, 501 N.E. 33rd St., P.O. Box A-931, Fort Lauderdale, Fla.

Quick setting mortar topping

Application of a quick setting mortar topping material to a rough and hazardous section of road surface on the Brooklyn Bridge was the means of saving the contractor two entire road-laying operations and considerable expense while making it possible to open that section to traffic the morning following application.

The material used for the job was Permatop, a hard surfacing adhesive product which is said to form an indestructible bond with concrete surfaces and cures within 8 hr.

The Brooklyn Bridge workmen applied the plastic-like material to a spalled area of several thousand square feet on the ramp approach to the Manhattan side of the bridge. The job was completed in 6 hr using only a surface trowel and a wood screed. Workmen gave a rough broom, skidproofing finish to the repaired surface.

The only requirements for an application of the topping is a clean, dry surface. Once a roadway is swept clean, the preparation of the topping can go ahead. This is done by mixing Permagile resin base with a liquid reactor and minutes later with a chemically treated aggregate. The result is a viscous substance that can be applied immediately.—Permagile Corp. of America, Woodside, Long Island, N.Y.

Instrument to measure mortar hardness

A cement and concrete laboratory instrument for measuring the rate of hardening of mortars sieved from concrete mixtures has been introduced by Soiltest.

Called the Mortar Penetrometer, the instrument uses penetration needles to determine the rate of hardening. Penetration resistance is measured by the force exerted to penetrate a mortar sample 1 in. The penetration force is measured by the compression of a calibrated spring inside the instrument; graduations are to 1 lb.

The device may be used to determine the effects of variables such as temperature, cement, mix design, additions, and admixtures on the hardening characteristics of concrete. The set complies with ASTM D 403.—Soiltest, Inc., 4711 W. North Avenue, Chicago 39, Ill.

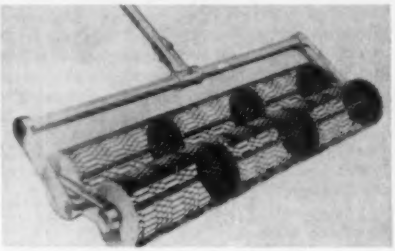
Vented portable space heater

Champion's Model 120 portable space heater features completely fume-free, odor-free operation according to the manufacturer.

Designed especially for use in confined areas where fumes are objectionable, the new Champ-Heater circulates 1885 cu ft of heated fresh air per minute. The unit burns kerosene or number one fuel oil and operates on standard 110 volt a-c. It has a 15-hr continuous burning capacity, fuel filter, high heat cut-off switch and automatic purging. A thermostat is available as optional equipment. — Champion Manufacturing Co., 3700 Forest Park Ave., St. Louis 8, Mo.

Finishing device

The Rollerbug tamps and levels a concrete slab in one operation. The specially designed rollers, mounted 10 in. on center push large aggregate at the surface of the slab down



into the mix about 1/32 to 1/16 in. Extension handles can be attached to the standard 12-ft handle so that an area 36 ft wide can be tamped.

The tool leaves a slightly keyed surface at the top of the mix for double course work, if desired. The roller pattern can be left in the slab for decoration and skid-proofing, if further finishing is not required.

The device is made of magesium and aluminum with only the bronze bearings and steel patterned roller surface made of other metals.—Goldblatt Tool Co., 1910 Walnut St., Kansas City 8, Mo.

Epoxy resin interior finish

Epo liquid coating is a pure epoxy resin formulated as a permanent interior finish for walls, ceilings, and structural metal. It is said to cure to a high, permanent gloss comparable in appearance and durability to ceramic tile finishes.

The coating is available in most decorator colors and can be applied by brush, roller, or spray. It adheres permanently to concrete, wood, plaster, brick, block, drywall, or metal according to manufacturer, and is highly resistant to chemical or abrasive action.—George E. Fines, Inc., 18640 Wyoming Ave., Detroit 21, Mich.

Thin gage flashing

Development of a thin-gage flexible sheet for elastic through-wall flashing and membrane waterproofing has been announced by Dow Chemical Co.

Trademarked Saraloy 200, the material is recommended for most waterproofing and flashing applications that do not involve direct sunlight. The material is described by Dow as a tough, flexible sheet, finely embossed on one side to provide mechanical keying action in motor joints, self-extinguishing, water and corrosion resistant, and easy to paint, to bond, and to fabricate. Nominal sheet thickness over-all is 1/32 in. Available in rolls 36 in. wide, weighing approxi-

mately 50 lb and containing 200 sq ft of material—The Dow Chemical Co., Building Products Sales, Midland, Mich.

Portable space heater

"Hot Shot" portable space heaters are now available in five models ranging in size from 75,000 to 450,000 Btu per hr. Equipped with sturdy handles and well balanced on wheels, the heaters are easily moved about and the heat directed where needed, according to the manufacturer.

They are said to be clean, safe, ventless, odorless, and economical to operate. — Kelley Machine Division, Wiesnes-Rapp Co., Inc., 285 Chicago St., Buffalo, N.Y.

Stressing washer

A new design of the stressing washer used in the Prescon system of post-tensioning prestressed concrete has been put into production. The threads on the washer have been replaced by one groove which means faster and easier attachment of the collar and pull rod of the hydraulic jack.—The Precon Corp., P.O. Box 4186, Corpus Christi, Tex.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

EXTERIOR WEATHERPROOF COATING—Flexible Resto-Crete, a weatherproofing coating for concrete and masonry, is described in this folder. Detailed information on its properties, wide range of colors available, and specifications are furnished.—Western Waterproofing Co., 1223 Syndicate Trust Bldg., St. Louis 1, Mo.

MARATHON CHEMICALS FROM WOOD (Bulletin No. 130)—Brochure contains products analyses and information related to the use of all of the lignosulfonates manufactured by Marathon for use in chemical processes and products.—Marathon, Division of American Can Co., Menasha, Wis.

ELECTRIC HEAT FOAM INSULATION VALUES CHART—To help the designer and builder of electrically heated buildings, The Dow Chemical Co. has published heat loss values for three of its plastic foam insulations. Each chart lists values by type of comfort application and thickness of foam. The values are expressed in both watts and *U* factors. Values charted are for Roofmate, for use under built-up roofs; Styrofoam, for insulation on masonry walls and in cavity walls; and Scorbord, for perimeter insulation. —The Dow Chemical Co., Midland, Mich.

MASONRY BLOCKS IN SCHOOLS—Brochure shows examples of school construction where concrete masonry was used. Discusses costs and values. —Concrete Masonry Association, 5205 Hollywood Blvd., Los Angeles, Calif.

HOME FOR THE AGED—(Flexicore Facts No. 89)—Six-page folder presents an example of high speed construction through the use of precast concrete floor and roof slabs with a cast-in-place structural frame. Photographs and detail drawings show the simplicity of the structural design.—The Flexicore Co., Inc., 1932 E. Monument Ave., Dayton 1, Ohio

ROOF MAINTENANCE PRODUCT CATALOG—New Horn Dri-N-Tite catalog is available describing plastic, primer, liquid and aluminum maintenance products for the patching, priming and coating of roofs of built-up composition, corrugated or sheet metal, slag, gravel, concrete or felt. Specifications, descriptions, uses, application, and coverage are included with particular information on the dispersant and wetting agent Horn 9C.—A. C. Horn Companies, Division of Sun Chemical Corp., 2133 85th St., North Bergen, N.J.

CONCRETE CURTAIN WALLS—An 18-minute full color-sound film highlighting the use of precast concrete panels for sheathing buildings has been released by the Portland Cement Association. The film illustrates nationwide use of concrete curtain walls in structures ranging from single-story commercial and industrial buildings to skyscrapers. It shows the wide latitude offered architects in choice of color, decorative designs, textures and panel shapes. Available on loan through PCA's 34 district offices located in major cities of the United States and Vancouver, British Columbia.—Portland Cement Association, 33 West Grand Ave., Chicago 10, Ill.

PROJECT: U. S. AIR FORCE ACADEMY, COLORADO SPRINGS (Bulletin P.74)—Concrete performance report presents a summary of information on the largest construction job in U. S. Air Force history. A total of 800,000 cu yd of ready-mixed concrete was designed, controlled, and placed under a wide range of conditions during construction. Report contains full details regarding the concrete work for the project. —The Master Builders Co., Cleveland 18, Ohio

THE STORY OF DURAZZO—Six-page multicolor brochure tells the story of Durazzo, a new terrazzo-type flooring that uses epoxy resins instead of cement. The booklet points out Durazzo's physical properties such as its quick curing, light weight, immunity to temperature effects, and resistance to stains and chemical attacks.—Durazzo, Inc., 623 River Rd., Garfield, N.J.

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